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Dynamic Soil-Structure Interaction Behavior on the Seafloor

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Foreword

Dynamic Soil-Structure Interaction (S-SI) behavior on the sea floor describes the coupling of a structure and the seabed and their combined response to the influence of waves and currents and the material properties of the sea floor. S-SI problems are significant to the Navy when structures placed on the sea floor must be recovered promptly, maintain position (such as no lateral movement), maintain stability (such as not bury or tilt), and not be affected by abrasion and/or sedimentation. This report develops the critical marine geotechnical, geological, and environmental problems of S-SI for structures placed on fine-grained sediments common to coastal areas. Only limited S-SI analysis of sand is presented in this report; however, significant additional work is seriously needed to support various naval operational scenarios requiring reliable predictive models. Detailed future research recommendations are provided herein and the ultimate success of the predictive models and operational strategies will depend critically upon the close integration of research by environmental scientists, geologists, and geotechnical engineers.

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Executive Summary

The dynamic response of a structure placed on the seafloor is governed by the influence of waves and currents and the material properties of the seabed. The coupling of a structure and the seabed and their combined response is described as soil-structure interaction (S-SI). Some S-SI problems have been analyzed by applying state-of-the-art principles and practices of marine geotechnical engineering and marine geology; however, other problems have not been analyzed adequately and require additional work. The marine environment is characterized by a variety of geological materials (ranging from rocks to muds) and hydrodynamic processes that make the analyses of S-SI problems complex. This report develops some of the more critical marine geotechnical, geological, and environmental problems of S-SI for structures placed on fine-grained sediments common to coastal areas. Only limited analyses of the S-SI behavior of sand is presented in this report, although significant additional work needed in this area is described and is strongly recommended.

Specific topics developed in this report include the following:

- bearing capacity and structure loading of the seabed
- prediction of structure penetration depth
- structure recovery or "breakout"
- significance of layered soils on S-SI
- skidding/lateral motion of structure on the seafloor
- scaling factors and their significance
- seabed response to surface wave activity
  - dynamics
  - seafloor failure/potential mass movements
  - soil strength degradation
  - environmental processes
- environmental data requirements/limitations of analyses
- critical topics for further work in the area of scour, "shakedown," scaling, layered soils, dynamic coupling of structure and seabed, and the development of predictive models
- recommendations

There is no reasonable substitute for reliable soils property data as input to models for predicting and analyzing S-SI problems on the seafloor. In the absence of such data, a "first-order" geotechnical engineering prediction of gross soils properties can be made, given a reasonable generic description of the geology and sediment types. The types of soil particles (grains), their size and percentage, the contained water, and the confining stress (related to depth of burial) are the most important elements of a soil that determine its bulk mechanical and physical properties. Without this basic information, little if anything can be rationally predicted in terms of soil behavior and its properties. Again, however, by knowing a general geographic location of interest where a structure will be placed on the seafloor, a geological interpretation of the soil types present and the environment can be made for most areas of the world. Because most S-SI analyses are complex and the nature of the

seafloor is highly variable, only approximate estimates of the behavior of a structure on the seabed can be made when limited data are available. Thus, rough estimates and predictions determined with the limited data should be carefully considered and used with the utmost caution.

Bearing Capacity/Penetration

At bearing pressures of approximately 100 psf, penetration and bearing capacity of a structure in cohesive soils is generally not a problem for static conditions. Some problems may be experienced, however, in soils with shear strengths of less than 25-30 psf. Bearing capacity and penetration into the seabed can be a serious problem for a structure sliding and/or rocking on the seabed. Wave and current forces can cause tilting of a structure, resulting in significantly higher bearing pressures on the seabed beneath the leading edge of the foundation. Although techniques are well known for analyzing bearing capacity for static situations, much less is known about and far fewer analytical techniques are available for analyzing bearing capacity and penetration for structures experiencing dynamic or cyclic loading. Such loading is a common problem in shallow-water areas. These problems require additional work performance tests, some of which can be performed in the laboratory.

Cyclic motion of a structure coupled to the seabed produces a process termed "shakedown." This dynamic process is expected to cause soil strength degradation and structure settlement or penetration in both cohesive and noncohesive soils. This process has not been studied adequately for the development of analytical models, but it is an important S-SI problem that influences the performance of structures that rest on the seafloor. Some of the more important unknowns restricting present modeling capabilities include such factors as scaling effects, penetration rate functions, dynamic forces, coupling phenomena, and strength degradation effects and the significance of these factors on the prediction and analysis of penetration and punch-through. Punch-through is a bearing capacity problem in layered soils where the bearing resistance of a stiff, upper-sediment layer is exceeded and the footing penetrates deep (punches through) into the underlying soft layer. The geological conditions conducive to punch-through (e.g., sands overlying soft clays) can occur in coastal environments as a result of changing environmental conditions and changes in source material. Thus coastal areas that appear to be sandy and have a stiff/rigid bottom can be unsafe for the placement of structures that rest on the bottom. A solution to these problems and questions would provide not only information relating to penetration and punch-through under repeated loading, but also a numerical solution to soil behavior (model), which can be used for calculation of stresses on the structure.

Breakout and Recovery

Breakout and recovery of a structure from a clean sandy bottom can be a problem if the structure has been scoured and/or if the structure has experienced "shakedown" due to cyclic motion. Generally, if neither condition

has occurred, and if no sand has piled up around the footings, recovery from a sandy bottom should not be a problem.

Breakout from muds and other soft clay bottoms can be a serious problem even with slight embedment. This problem can occur at embedments of only a few inches. In all likelihood, some assistance will be required for breakout from soft muddy bottoms where penetration has been minimal. Increasing burial and penetration by a structure may be experienced due to scour and/or dynamic soil-structure interaction common in coastal environments.

Skidding/Lateral Motion

Lateral movement or skidding of a structure sitting on the seafloor is a result of waves and currents acting on the structure. The extent of the motion will depend not only upon the forces acting on the structure, but also upon the sediment lateral resistance to skidding and the slope of the seafloor. The lateral resistance depends upon the friction between the base of the structure and the sediment (cohesionless sediment) or on the adhesion between the base of the structure and the sediment (cohesive sediment). If the structure penetrates the sediment, then the S-SI process becomes more complex. Simple formulas can predict lateral resistance for simple case situations; however, no solutions to the problem are available for cases involving repeated cyclic loading (dynamic effects). Essentially little-to-no data are available on the undrained shear strength of the upper 6 inches of soft, weakly cohesive coastal muds and on the strength characteristics of noncohesive sediments subject to very light normal (vertical) loads. Since the resistance to lateral motion depends in large part upon the properties of the surficial soils (upper few inches), reliable predictions of lateral movement are difficult to make because of this fundamental lack of data and information on the process. Additional investigations are required. Essentially no analytical solutions are available for predicting lateral sliding on stiff clays. Also, additional work is required that would include rate effects on resistance to movement.

Mass Sediment Movements

In most cases mass movement (i.e., slumping, slope failure, landslide, or flow slide) of submarine sediment will not be triggered by placing a structure on the seabed. Locations where mass movements might be initiated include edges of escarpments, steep slopes or metastable soils, such as some silts and fine sands (as occur in fjords). Most offshore mass movements are a function of environmental conditions and are triggered by storms and earthquakes. Mass sediment movement, such as a flow slide, can result in a structure sinking into the disturbed sediment mass and being virtually impossible to recover. On soft mud or clay seafloors, storm waves can cause significant motion of the seabed even though sediment shear failure does not occur. A structure coupled to a soft seafloor can experience severe dynamic motion under wave

forces. These motions can be predicted with adequate sediment data and a knowledge of the environmental conditions.

Recommendations

Those mechanisms of soil-structure interaction that are not well understood should be evaluated in detail as a first step toward providing better predictions of performance of a seafloor-resting structure. The following summary provides a brief description of the work that should be initiated during the next two years to support the planning of Navy operational scenarios.

• Dynamic bearing capacity and "shakedown" for structures rocking and sliding on the seafloor under the influence of waves and currents requires improved description and modeling. A solution of the common problem of soil-structure interaction in layered soils is required. At present, no adequate numerical solutions to soil behavior (models) are available for handling the dynamics for layered soils in the area of bearing capacity, penetration, and breakout, particularly for seafloors characterized by sands overlying soft clays.

• Analysis of field and lab data and model development to better predict structure motion and skidding on soft muds and on stiff clay bottoms is of high priority. Meager data are available on the undrained shear strength of the upper few inches of soft muds and the strength characteristics of non-cohesive, granular sediments subjected to very light vertical (normal) loads. All available data should be evaluated and compiled. Additional work is required addressing the importance of rate effects on the resistance of a structure to movement on the seabed.

• The scaling effects and factors, penetration rate functions, dynamic forces, coupling phenomenon, and strength degradation time effects for punch-through and breakout in layered soils, including sands overlying soft clays, must be determined. The significance of these factors on predictive model development should be determined.

• The loading history of the structure on the seafloor, including current and wave dynamic loading, and uplift load during breakout, must be assembled and verified.

• The probable structure settlement/penetration in a range of seafloors (sediment types) should be quickly examined to identify those seafloor environments that pose an operational problem. Also, those seafloor areas where it is desirable to deploy a structure, but not possible because of the cohesive sediment breakout problem, should be identified. Given this identification, then, better assessment can be made of the desirability of improving our predictive capability for breakout force and of the desirability of developing breakout aids, such as water jetting assistance.

• The scaling factors involved in the process of scour for the subject footing(s) (specified geometry and bearing pressure) under the influence of waves and currents must be determined.

— The process of scour and fill about a structure on cohesionless seafloors, including the effects of structure sliding and sediment plowing, must be evaluated

to develop a prediction of depth of sand infill against a structure for various operating environments and scenarios. The sand infill height must be known in order to determine if breakout from sand and gravel seafloors will be a problem.

— The structure foundation must be sealed in those areas in contact with the seafloor and in the sand transport zone to prevent the entry of sand into the foundation structure, as this sand fill seriously reduces the available uplift force for breakout. Assessment should be made of these factors.

• A technical assessment of the geology and soil types should be completed for all general operational areas. With this information and background, some judgment and prediction of the structure performance can be made.

• A working base of data should be assembled on the properties of weakly cohesive submarine soils (e.g., the upper few inches of mud seabeds) and soil properties and strength characteristics of lightly loaded, noncohesive, granular soils.

Acknowledgments

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Contents

Symbols and Definitions

A: Bearing area of a structure on the seafloor *(L^2)* or area of footing *(L^2)*

A_c: Contact area of the leading edge of the footing *(L^2)*

A_s: Area of the base in contact with the sediment *(L^2)*

a: Adhesion to the base material of the sediment *(F/L^2)*

B: Footing width *(L)*

D: Depth of bearing surface below sediment surface (for breakout equations, D is in feet), or pipeline diameter *(L)*, or depth of embedment *(L)*

d_c, d_q, d_γ: Correction factors for depth of base embedment (unitless)

d_g: Sediment grain diameter *(L)*

E_i: Initial slope of the stress-strain curve *(F/L^2)*

F: Horizontal force acting on structure *(F)*, total force per unit length of pipe *(F/L)*

F_B: Buoyant force per unit length of pipe *(F/L)*

F_b: Design or allowable force available for breakout after applying Factor of Safety *(F)*

F_f: Friction force on the sidewall of a structure *(F)*

F_q: Bearing capacity of sediment under a vertical load *(F)*

F_s: Suction force resisting breakout between base of structure and underlying sediment *(F)*

F_{Ib}: Immediate breakout force carried by the sediment *(F)*

F_{Lb}: Long-term breakout force carried by the sediment for time t_b *(F)*

$F.S.$: Factor of Safety (unitless)

f_f: Friction stress on the sidewall of a structure *(F/L^2)*

f_{fD}: Friction stress acting on the sidewall of a structure at depth D *(F/L^2)*

G: Shear modulus *(F/L^2)*

G: Viscoelastic secant shear stiffness

G_1: Viscoelastic secant shear stiffness for given vane rotation rate at time of 1 sec

g_z: Vertical force coefficient for both push down and pull up on horizontal cylindrical structures (pipelines)

H: Depth of footing embedment into the sediment *(L)*, wave height *(L)*, thickness of upper soil stratum *(L)*

h: Depth below seafloor surface in slope stability analyses *(L)* depth of pipe below surface of sediment *(L)*

i_c, i_q, i_γ: Correction factors for inclination of resultant load (unitless)

K: Earth pressure coefficient (unitless)

K_s: Coefficient of punching shear (unitless)

k: Bernstein Modulus for sinkage, a function of size and shape of plate *(F/L^3)*

k_c: Soil constant (Bekker Modulus of sinkage in cohesive soils)

k_ϕ: Soil constant (Bekker Modulus of sinkage in cohesionless soils)

L: Length, wavelength, contact length of the footing (leading edge) *(L)*

N_c: Bearing capacity factor for soil cohesion (unitless)

N_q: Bearing capacity factor accounting for overburden pressure (unitless)

N_R: Reynolds number for model tests (unitless)

N_γ: Bearing capacity factor accounting for sediment friction angle during drained shear (unitless)

n: Sinkage exponent, a constant for the given soil (unitless), viscoelastic rate exponent for the sediment (unitless)

n_o: Slope of viscoelastic stiffness versus time on log-log plot (unitless)

P_p: Passive pressure or soil resistance mobilized per unit length of footing sidewall *(F/L)*

p: Average breakout pressure applied to sediment in psf

Q: The total load *(F)*

q: Overburden pressure, γz *(F/L²)*

Q_l: Total lateral load resistance *(F)*

q_d: Vertical stresses induced in sediment by a footing bearing surface *(F/L²)*

q_f: Ultimate unit bearing capacity (F/L²)

q_s: Bearing pressure causing sinkage (F/L²)

R: Resulting force acting at structure bearing surface (F)

S_u: Undrained shear strength (F/L²)

S_{ub}: Undrained shear strength of the bottom soil stratum (F/L²)

S_{ut}: Undrained shear strength of the top soil stratum (F/L²)

s_c, s_q, s_γ: Correction factors for shape of base (unitless)

T: Breakout time parameter (unitless), torque *(LF)* wave period (sec/cycle)

T_n: Normalized torque from vane shear test *(LF)*

V: Relative velocity between the pipeline and the sediment in the free field *(L/T)* water particle velocity parallel to the sediment surface *(L/T)*

W: Submerged weight of structure *(F)*, total vertical force applied to the footing *(F)*

Z: Depth of sinkage *(L)*

γ_b: Effective or buoyant unit weight of the sediment *(F/L³)*

$+\Delta p$: Pressure increase beneath wave crest *(F/L²)*

$-\Delta p$: Pressure decrease beneath wave trough *(F/L²)*

δ: Surface friction angle on the side of structure (degrees)

ε: Axial strain (unitless)

ε_f: Axial strain at failure (unitless)

θ: Vane rotation (degrees)

θ_d: A particular vane rotation angle (degrees)

$\dot{\theta}$: Vane rotation rate in viscoelasticity tests (degrees/T)

λ: Wave length *(L)*

μ: Viscosity of seawater (FT/L^2)

ϱ: Density of seawater (FT^2/L^4)

σ_1: Axial total stress applied to the specimen in a triaxial test (F/L^2)

σ_3: Lateral total stress applied to the specimen in a triaxial test (F/L^2)

ϕ: Friction angle (degrees)

$\bar{\phi}$: Effective angle of internal friction (degrees)

Dynamic Soil-Structure Interaction Behavior on the Seafloor

1.0 Statement of Problem

The U.S. Navy operates in virtually all types of oceanic environments, characterized by a wide range of climatic conditions and oftentimes rapidly changing sea states. The oceanic water masses are uniquely coupled to the seafloor through a variety of geological and hydrodynamic processes critical to naval operations. Because of the critical need to predict the complex seafloor processes and changing environmental conditions with reliability, the Naval Ocean Research and Development Activity (NORDA) was tasked to provide specific environmental soil-structure-interaction (S-SI) analytical support to the David Taylor Research Center (DTRC). This report provides preliminary geotechnical engineering analyses of specific S-SI processes and phenomena to be used to establish reliable and predictable operational scenarios.

The fundamental S-SI problems addressed involve the placement and recovery of structures on the seafloor that are subject to the dynamic influence of waves, currents, and the seabed as a function of the material (soil) properties and related environmental conditions and processes. In shallow-water environments the seabed interacts very dynamically with the overlying water column due to wind-generated waves, and the response of a structure to the dynamic coupling of the water-structure-seabed interaction is strongly dependent upon the seafloor and subseafloor soil properties. NORDA was tasked to work during FY86 primarily on problems associated with fine-grained soils (clays); however, some limited (preliminary) analyses are presented on the dynamic behavior of sands (cohesionless materials).

The analytical solutions to the above problems are reasonably complex. Major factors critical to obtaining reliable operational solutions to structure penetration, settlement, movement, and recovery (breakout) are the direct availability of reliable soils property data, reasonable estimates of seafloor sediment variability, and predictable wave climates (surface wave activity and duration). To minimize the time required to obtain analytical solutions to the various problems investigated and reported herein, NORDA has utilized capabilities at Texas A&M University (TAMU), Department of Civil Engineering, in particular, the state-of-the-art geotechnical computer models and techniques developed by TAMU. These models and techniques were developed by TAMU for the offshore petroleum industry, which has been confronted with similar S-SI problems for several years. This report therefore represents a synergistic effort by NORDA and TAMU.

1.1 Fundamental Considerations

The seafloor in shallow-water coastal areas is commonly quite dynamic, and structures resting on or within the surficial sediments are subject to dynamic motion due to oscillatory forces from currents and waves. Other environmental processes, such as internal waves, earthquakes, tidal changes, and mass movements (bottom failure) of the seafloor can cause motion of a structure. In addition, scour around structures resting on the bottom is quite common and can lead to undercutting of foundation pads and tilting of bottom-resting structures. Depending upon seasonal climatic conditions and the degree of sediment transport, the structure can be buried, abraded, and transported in dynamic coastal environments. These structures are sited on the seafloor in water depths from 300 to 600 ft where they are subject to significant wave and current forces. The structures we are dealing with apply maximum static vertical loads only 10 to 25% of those applied by lighter offshore structures, and potential vertical and horizontal dynamic loads are at least as high as (or higher than) the static load. Thus, the static vertical loading is very light, while the dynamic wave and current loading are proportionately quite high and the structure will slide about and rock on the seafloor when subjected to storm conditions.

Sinkage of the structure into the seafloor sediments, as well as lateral sliding and rocking, adversely impact on the performance of the structure. In addition, breakout of the structure from the seafloor sediments, after sinkage due to overloading of soft sediments or due to scour of supporting sediments, can be a serious

problem because of the limited uplift force available for breakout.

Sinkage of a structure into the seafloor can result from a number of mechanisms. One is simple vertical displacement with the structure penetrating into the seafloor and displacing sediments that are heaved up beside the structure (Fig. 1.1-1). Addition of a lateral load as from a current would result in an eccentric loading on the structure base yielding the nonsymmetric penetration and sediment heave depicted in Fig. 1.1-2. The mechanics of predicting these penetrations are discussed in Section 2.2. Given the structure base size and known loading, these penetration mechanisms are not expected to occur in operating environments save for soft mud seafloors as found on rapidly building river deltas and in estuaries. More commonly, because of the very low static vertical loading, the failure mechanism will be one of lateral sliding under combined current and wave loading. The mechanics of predicting the sediment resistance to lateral sliding are discussed in Section 2.4. When the current force predominates, the sliding will be unidirectional (Fig. 1.1-3) and probably intermittent as a series of "hops." When wave forces predominate, the sliding will be oscillatory (Fig. 1.1-4), with the oscillation distance being a function of the resisting forces developed by the sediment "berms" and the period of the driving ocean waves. As the wave forces alleviate, it appears likely that the lateral sliding motion will transition to a rocking motion of the structure (Fig. 1.1-5) and shakedown of the foundation, either through erosion of supporting sediment by the flushing action of trapped water at the edges of the rocking foundation or through softening of a cohesive sediment due to remolding under the repetitive loading and flow from beneath the foundation.

At this time, penetration of the foundation due to scour (Fig. 1.1-6) is not known to be a problem for the structures in question. Experience suggests that scour and subsequent settlement is a likely penetration mechanism. This mechanism should be borne in mind and operating personnel advised of its likely occurrence. While scour and subsequent penetration itself may not pose an operating hazard to the system unless penetration exceeds certain allowable values, subsequent sediment infill and breakout from that infill condition are most certainly a potential serious problem. Breakout of the structure from the sediment in the process of recovery (Fig. 1.1-7) can be a serious problem if penetration of the foundation is significant. The meager techniques available for predicting necessary forces to accomplish breakout are discussed in Section 2.3. Breakout is recognized as a significant problem for the structure in question, because the force used to accomplish breakout must be severely limited to reduce the potential for uncontrolled ascent of the structure after breakout.

An outline of the common environmental processes that characterize coastal areas is given in Table 1.1-1. Also the local processes occurring in proximity to the structure and important to the prediction of structure behavior and its motion on the seafloor are summarized in Table 1.1-1. These factors are normally considered carefully when structures are to be placed on the seafloor. Reliable predictions of S-SI behavior can be made with high confidence only when adequate soils and environmental data (water depth, wave, and current conditions) are known and carefully projected with time. Soil properties and environmental data normally required for making adequate predictions of offshore structure performance are summarized in Table 1.1-2. For this application none of the sediment properties data required for foundation design will be available. Available data will probably be limited to the following:

- acoustic and visual observations of seafloor texture
- possibly an acoustic high resolution subbottom profile
- a record of the resistance offered by the sediments to penetration of a free-fall probe over the upper 3 ft
- possibly a sample of the surficial sediments for visual classification

One purpose of this project is to identify environmental conditions, especially seafloor conditions, under which the structure can be sited reliably and the structure motion performance and breakout load can be predicted within allowable magnitudes. As the availability of quality data decreases, the reliability of predicting structure behavior under well-understood mechanisms also decreases. If the mechanisms governing behavior are poorly understood, then behavior predictions that would be poor even with good data, will be totally unreliable with poor data. Consequently, those mechanisms which are not well understood should be studied in detail as a first step toward providing better predictions of behavior when limited seafloor data are available.

1.2 Scope and Purpose of Report

This report develops the geotechnical engineering aspects of S-SI on fine-grained (clayey) seafloor soils in water depth common to coastal environments. Because of directives received from DTRC, funding, and time constraints, only limited analyses were completed for S-SI behavior of sands (noncohesive material). The topics developed herein include problems associated with bearing capacity and loading of the seabed; seabed response to surface wave activity and seafloor failure (environmental processes, soil strength degradation and potential mass movement); prediction of structure penetration depth; structure recovery or breakout; and preliminary analysis of the influence of layered soils on S-SI analysis.

The remainder of this report summarizes future required efforts and investigations that are prerequisites to establishing reliable operational guidelines for naval applications.

Table 1.1-1. Potential S-SI problems for consideration.

Environmental Processes (mass processes)

- Bottom failure (without structure on bottom)
  - Environmental forcing functions, e.g., waves, earthquakes (seismic shock), internal waves
  - Scour-oversteeping by sedimentation (may be seasonal), bioerosion, iceberg keels
  - Strength decrease: pore pressure increase due to:

    waves
    osmotic pressure changes
    biogenic methane production
  - External, man-induced: ships, construction activities, weapons effects (shock waves, etc.)
  - Tide-induced flow slides (sands / silts)
  - Collapse of bottom due to environmental conditions (little or no translation)
- Nepheloid layer (high-density bottom water)
- Sand wave migration due to storms
- Changes in water column characteristics due to differences in bottom characteristics (properties), i.e., wave degradation characteristics, water velocity, pressure

Processes Due to Structure on Bottom (localized processes)

[structure configuration (effects of) and changes produced by currents and waves]

- Scour: sand/silt/clay scour resulting in the following:
  - settling
  - tilting
  - movement
  - burial, differential settling
- Localized strength degradation and pore pressure changes due to repeated loading (cyclic loading of structure on bottom)
  - thermal gradients (frozen ground/permafrost) freeze-thaw
- Bottom failure/bearing capacity)
  - initial failure and failure due to strength degradation
  - prediction of penetration depth
  - breakout forces required
- Settlement -- consolidation
- Prediction of skidding and sliding

Table 1.1-2 Data requirements for S-SI analysis.

Soil Properties (*required for all stratigraphic units*)

- Noncohesive sediment
  - Grain size (mm)
  - Specific gravity and water content
  - Bulk density
  - Angle of internal friction (on effective stress basis obtained from direct shear or triaxial tests)
  - Permeability
  - Relative density
- Cohesive sediments
  - Grain size (mm)
  - Specific gravity and water content
  - Atterberg limits (liquidity index)
  - Bulk density
  - Undrained shear strength (by miniature vane or unconfined compression - UU)
  - Remolded strength/sensitivity
  - Consolidation and permeability data
  - Consolidated undrained shear strength (on effective stress basis with pore pressure measurements, CU - test)

Environmental Data (required for all sites)

- Bottom slope
- Wave climate and currents
- Water depth
- Water density (salinity)
- Bottom roughness

Structure Data

- Size, shape, and weight
- Footprint/configuration/shape
- Static and dynamic bearing pressure on footings (secondary vibrations)
- Influence of structure on currents and waves around footings

***In Situ* Data**

- Cone penetrometer resistance
- Pore pressures
- Vane shear strengths
- Resistivity/conductivity

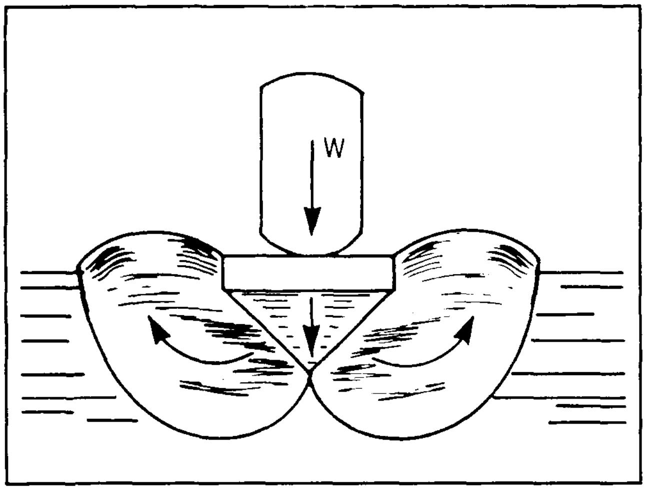


Figure 1.1-1. Bearing capacity or sinkage failure of footing on cohesive sediment, vertical noneccentric loading. W = submerged weight of structure.

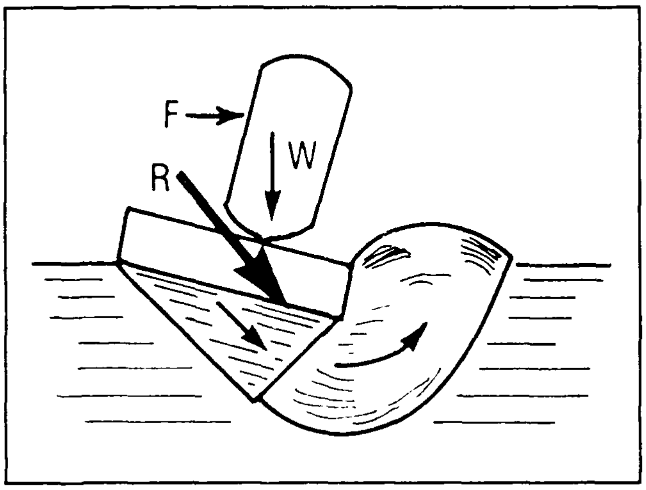


Figure 1.1-2. Bearing capacity or sinkage failure of footing on cohesive sediment, inclined eccentric loading. F = horizontal force acting on structure; R = resulting force acting at structure bearing surface.

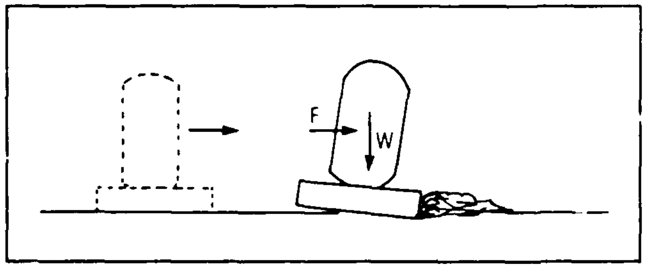


Figure 1.1-3. Lateral sliding and development of passive wedge or berm, unidirectional lateral force.

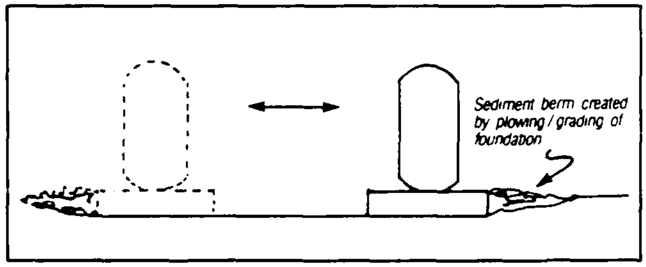


Figure 1.1-4. Oscillatory lateral sliding subject to strong wave forces and development of motion-limiting berms, strong oscillatory lateral force.

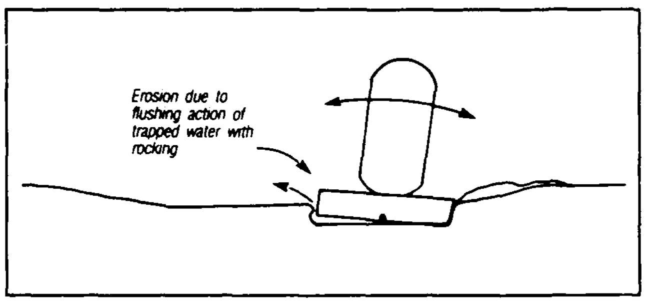


Figure 1.1-5. Sinkage/embedment of footing while rocking subject to oscillatory lateral force.

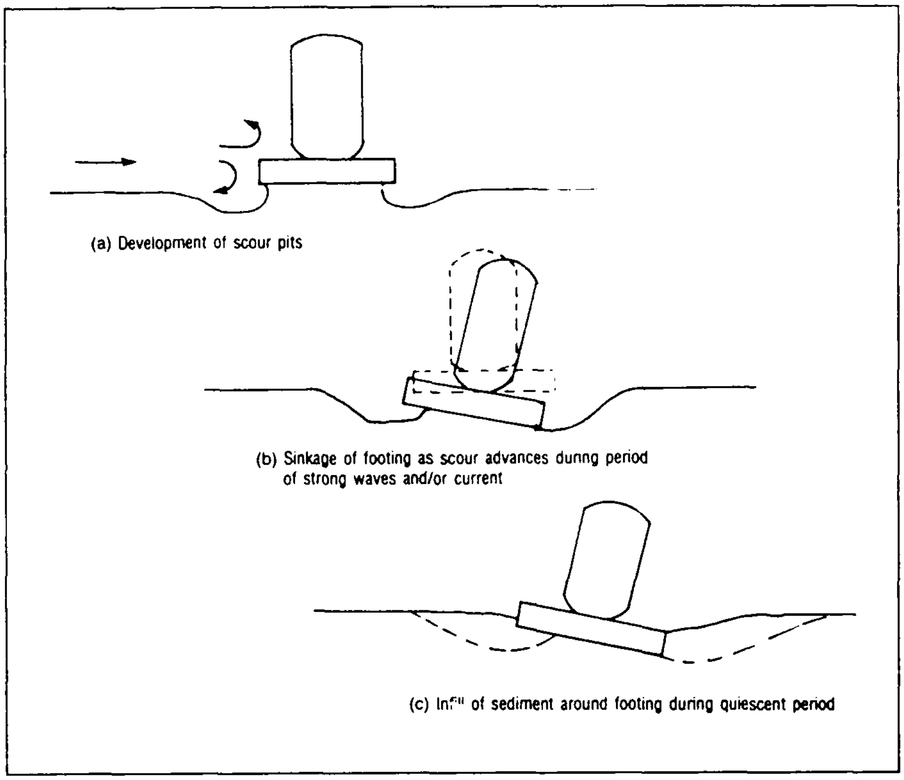


Figure 1.1-6. Sequence of scour and infill.

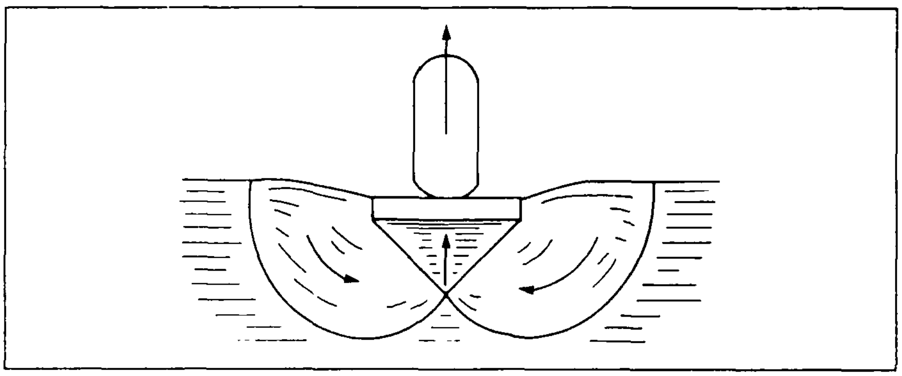


Figure 1.1-7. Breakout of footing from seafloor sediments, reverse of bearing capacity or sinkage.

2.0 Penetration, Breakout, and Lateral Resistance

2.1 Definitions

2.1.1 Penetration. For this report, penetration is defined as the sum of all mechanisms causing or leading to downward displacement of a structure after first contact with the initial seafloor surface. Penetration mechanisms can be grouped into three categories: penetration arising due to shear failure of supporting sediments, penetration due to elastic deformation and time-dependent volume change of the sediment usually referred to as "settlement," and penetration due to removal of sediment supporting a structure either by scour or by "pumping" of the bearing surface.

2.1.1.1 Penetration by soil shear failure. A structure placed on a sediment surface will penetrate into that sediment until the shear strength of the sediments along the governing shearing surfaces is sufficient to support the bearing load. Penetration due to shear failure may resume if the bearing load on the sediment is increased at a later time as from current or wave loading on the structure, or penetration may resume due to a decrease in sediment strength caused by sediment remolding or liquefaction under wave or earthquake loading.

2.1.1.2 Settlement. Settlement is that portion of the structure penetration occurring without shear failure of the sediment and without removal of sediment as by scour. Settlement includes three components. The first is initial settlement due to the plasto-elastic deformation of the sediment occurring immediately with load application without sediment volume change. The second is the primary consolidation settlement, in which the sediment structure is responding to the loading much like a sponge, with deformation rate governed by the hydrodynamic lag of the pore water being squeezed out of the sediment pores. The third is the secondary consolidation settlement, which involves continued deformation of sediment skeleton but at a relatively slow rate where the hydrodynamic lag of the pore water expulsion no longer influences settlement. The probable settlement penetration of the structures of this study has not been calculated. Settlement of the structures on cohesionless (sand) seafloors is expected to be negligible; settlement on soft, cohesive (mud, soft clay) seafloors is expected to be sufficient, especially with wave force loadings, to lead to a problem in achieving breakout (Section 2.3.3.2). The probable settlement penetration on a range of seafloors (sediment types) should be examined to identify those seafloor environments that may pose an operational problem.

2.1.1.3 Scour or pumping. This penetration mechanism is by far the most difficult to predict, especially its rate of progress. Scour is that process where sediment grains are eroded from the seabed by wave- and current-induced water motions aggravated by the structure as an obstruction in the flow field. Formation of a scour pit beside and possibly undercutting the edge of the structure removes sediment providing bearing support, and the structure tilts or slumps toward the scour pit. Pumping of sediment from beneath a structure results when the structure rocks subject to the oscillatory loading of the waves with portions of the base lifting off the seafloor. When the base comes back down on the seabed, the water pumped from between the base and the sediment will normally exit with sufficient velocity to erode sediment beneath the base, thus aggravating the rocking problem. Penetration due to scour and pumping is a very difficult problem to analyze with most of our predictive capability being empirical and requiring modeling tests for quantitative prediction. We have not attempted to predict penetration due to scour and pumping herein.

2.1.2 Breakout. Breakout is that process of freeing a structure from its embedded position in the sediments. Breakout involves overcoming frictional forces between the sediment and sides of the embedded structure, and suction forces acting on the base. Breakout is reviewed in Section 2.3.

2.2 Penetration

2.2.1 General. A structure placed on the ocean floor will usually penetrate or sink into the sediment. Penetration or sinkage usually involves large movements that result from sediment shear failure below the loaded area and large displacements of the sediment. In the classical foundation engineering sense, designs are developed to eliminate sinkage failures by ensuring adequate safety factors, to eliminate shear failure beneath the footing, thus holding penetration or sinkage to negligible magnitudes. Sinkage is important in some problems, however, such as trafficability of vehicles and the problem at hand involving a structure on the ocean bottom. For these problems two methods have been used to predict the depth of penetration into sediments. The first is an empirical sinkage theory from trafficability work. The second is a reverse application of the bearing capacity theory where trial calculations are made for different penetration depths to find the depth at which shearing resistance of the soil balances the applied bearing pressure.

2.2.2 Prediction Theories

2.2.2.1 Sinkage theory. Plate sinkage tests are often used to evaluate sinkage and to obtain information for scaling up to prototype conditions from model tests. Several investigators have used the following formula to evaluate plate sinkage tests (see Fig. 2.2-1)

$$q_s = kz^n , \qquad (2.2\text{-}1)$$

where $q_s = Q/A$ = the average unit pressure (F/L^2),

Q = the total load (F),

A = area of footing (L^2),

n = the sinkage exponent, a constant for a given soil (unitless),

k = sinkage modulus, a function of size and shape of the plate,

z = the depth of sinkage (L).

The results of sinkage tests on different sizes of footings can be portrayed as shown in Figure 2.2-2.

Bekkar (1960) proposed a relation between k and the plate width B as

$$k = k_\phi + k_c/B \,, \qquad (2.2\text{-}2)$$

where k_ϕ *and* k_c are soil constants termed the Bekker modulus of sinkage. k_ϕ is dominant in cohesionless soils and k_c in cohesive soils. Figure 2.2-3 illustrates a graphical method for the determination of the two modulus values when several sets of B and k are available. Although the sinkage equation has great appeal owing to its simplicity and the possibility of using it to scale model results up to prototype, several difficulties are associated with its use in practice. Hvorslev (1970) reports on the results of several well-performed tests on clay and sand where the data do not plot as linear relationships, as indicated in Figure 2.2-4, and in some cases the lines for different size plates actually cross each other. These results indicate that the coefficient n is a function of plate size, as well as soil properties. As such, it is difficult to place confidence on the use of model tests to predict prototype sinkage. Another potential problem is that the sinkage equation was meant for use in homogeneous soils. In situations where the soil changes strength with depth, model plate tests will not induce stresses in a deeper soil layer which may have lesser strength, although a larger prototype footing would. This situation is shown in Figure 2.2-5. Also, there is no place in the sinkage equation for the effects of inclined or eccentric loads or for the effects of repeated loads. Based on his analysis of the sinkage equation, Hvorslev suggests that the general bearing capacity formulas may be better suited for evaluating sinkage.

2.2.2.2 Bearing capacity theories. For a flat surface loaded vertically, the general bearing capacity equation is

$$q_f = S_u N_c + q N_q + 0.5\,\gamma'\,B N_\gamma \,, \qquad (2.2\text{-}3)$$

where q_f = ultimate unit bearing capacity (F/L^2),

S_u = undrained shear strength (F/L^2),

q = overburden pressure, $\gamma z'(F/L^2)$,

γ' = buoyant unit weight of sediment for submerged condition (F/L^3),

B = footing width (L),

N_c, N_q, N_γ = bearing capacity factors (unitless).

The bearing capacity factors for soil cohesion, overburden pressure, and friction, N_c, N_q, and N_γ, respectively, have been developed by several investigators. In general, the differences between the values proposed by various investigators are small, usually amounting to less than 20% difference in the ultimate bearing capacity. Curves are presented in Figure 2.2-6 for the more generally recognized values of the bearing capacity factors. These factors are obtained for strip footings, resting on the soil surface and subjected to vertical loads. Modifying factors have been developed to take into account other shaped footings, footings embedded below the ground surface, and inclined or eccentric loads. These give rise to a more general bearing capacity formula:

$$q_f = S_u N_c s_c d_c i_c + q N_q s_q d_q i_q + 0.5\,\gamma'\,B N_\gamma s_\gamma d_\gamma i_\gamma \,. \qquad (2.2\text{-}4)$$

where the modifying factors for shape, depth, and eccentricity or inclination are indicated as *s*, *d*, and *i*, respectively.

In case of rectangular footings, the shape factors are (Hvorslev, 1970)

$$s_c = 1.0 + 0.2B/L \qquad (2.2\text{-}5a)$$

and

$$s_q = 1.0 - 0.2B/L. \qquad (2.2\text{-}5b)$$

The value of s_γ is not so clear, but one recommendation is (Hvorslev, 1970)

$$s_\gamma = 1.0 + B/L\sin\phi \,, \qquad (2.2\text{-}5c)$$

where ϕ = effective angle of internal friction.

One major objection to the use of the general bearing capacity formula is that the modifying factors *s*, *d*, and *i* have been determined separately without considering the influence or interaction of the other factors. For example, the failure surfaces developed under inclined loads may not coincide with those developed under purely vertical loads. Other problems will be discussed later.

In the theoretical development of most bearing capacity formulas, it has been assumed that soil adhered perfectly to the footings, i.e., the footings were rough. Subsequent studies have shown that the footing roughness has little effect on the bearing capacity in clays. In sands, a frictionless footing yields smaller bearing capacities than a rough base, although the base must be very smooth to produce a significant decrease in bearing capacity.

Tilting of the footing also can cause a decrease in the bearing capacity, and the decrease becomes more

significant as the angle of tilt increases. A tilt of 10° from the horizontal would decrease the bearing capacity of a clay only 5% to 6%, but the same tilt could cause a 20% to 25% decrease in bearing capacity in a sand according to Meyerhof (1953).

Horizontal footings resting on sloping soils will have a decrease in bearing capacity from those resting on a horizontal surface. For shallow-angle slopes, this decrease will amount to less than 10%. If the footing is also tilted, there will be either an increase or decrease in the bearing capacity depending on the direction of the tilt.

As stated earlier, one of the major problems with the general bearing capacity theory is that the modifying factors were developed more or less independently of each other, and their interacting effects for the most part are unknown. More important in this case is the fact that the general bearing capacity equations were developed for soils that are generally considered to be competent or strong enough to be considered for foundation soils. Very weak sediments, those with undrained shear strengths of 500 psf or less, were not considered separately, but indications are that they should be. The bearing capacity theories generally start with the assumptions made by Prandtl that a surface of soil failure is developed beneath the footing as shown in Figure 2.2-7. This is often referred to as the case of general shear. With very weak soils, there may be incomplete development of the shear planes and a punching failure may occur. Terzaghi (1943) termed this as local shear and suggested that the bearing capacity for weak cohesive soils should be reduced by two-thirds to take this into account. Insofar as can be determined, this has never been verified experimentally.

The bearing capacity theory also does not consider creep of the sediments. In normal competent soils this is not a significant problem; it is usually minor compared to soil consolidation. But with weak marine sediments, creep may be a substantial portion of the overall downward movement. Tests at TAMU have shown that soft marine sediments definitely exhibit the properties of viscoelasticity, and that creep under load can be expected.

One final concern of the bearing capacity theory involves the application of a horizontal or tangential stress to a footing. The theory shows that when the horizontal stress at the contact surface approaches the shear strength of the soil, the bearing capacity of a purely cohesive soil is reduced by as much as 50% and for a cohesionless soil the bearing capacity becomes almost zero (Hvorslev, 1970).

2.2.3 Field Verification of Penetration Prediction Theories

2.2.3.1 General. A review of the literature has failed to produce any evidence of the use of the basic sinkage equation (Section 2.2.2.1) for predicting sinkage of footings in soft marine sediments. Since the purpose of any foundation design is to prevent sinkage, there is also little information available on the suitability of the bearing capacity formulas to accurately predict sinkage. However, a significant contribution in this respect comes from the offshore industry as a result of the problems that have been encountered with jack-up drilling rigs. Numerous jack-up rigs have suffered catastrophic failures during the jacking up process and others have failed while being subjected to forces during storms (Young et al., 1984). Analyses of the failures provide some evidence of the suitability of the bearing capacity formulas for predicting sinkage, at least for soft sediments.

Two basic types of jack-up rigs have been used in the offshore industry. Mat-supported rigs have several legs connected to a horizontal mat that rests on the seafloor. In operation the mat is jacked into the seafloor until the mat entirely supports the drilling platform. Then the platform is jacked above the water level. Most rigs of this type have A-shaped mats, which somewhat complicates the calculations of the depth of penetration. Average bearing pressures range from about 400 to 700 psf. (These are significantly larger than the 100 psf considered for our object.) Rig designers advise that the mat should not penetrate more than the mat thickness, which varies with the size of the rig, but usually does not exceed 10 ft. Because of the low bearing pressure, this value is seldom exceeded, even in very low-shear-strength sediments.

The other type of platform has three or more legs with large footings or "spud cans" at the bottom of each leg. These legs are independently jacked into the seafloor. The combined area of the spud cans is much smaller than the mat area on mat-supported rigs, and the bearing pressures may be 10 times higher. As a result, in soft sediments the legs may penetrate 60-90 ft.

Jack-up rigs will be subjected to additional forces during storms, and these storm forces can cause further settlement, or even failure by overturning. For this reason, the platforms are "preloaded" during installation by pumping ballast water into the rigs to produce a temporary load about 25% greater than the normal working load.

Owing to the significant consequences of failure, many rig operators now require a geotechnical site investigation to be performed prior to setting up a jack-up rig. The most important information obtained from a geotechnical site investigation is the profile of sediment shear strength with depth, and to a lesser extent, the unit weight of the sediment with depth. Since the depth of penetration is not known a priori, the borings are usually taken to a depth significant enough to ensure safety. In recent years, the shear strengths have been obtained with in situ measuring devices, usually the in situ vane shear apparatus (Ehlers et al., 1980).

2.2.3.2 Cohesive soils. Young et al. (1981, 1984) have provided analyses of several rig failures in clays and also a comparison of predictions and actual performance where leg penetrations were measured. Most of the results were obtained from rigs operating in the Gulf of Mexico, although some other locations are represented. If the sediment shear strength is fairly uniform with depth, Skempton's (1951) bearing capacity coefficients (Fig. 2.2-6) appear to adequately predict the depth of sinkage.

An important aspect of the penetration calculations is knowing the depth to which the significant stresses imparted by the footing will extend, as this is the depth in which the shear strength of the sediment is important. Skempton (1951) originally recommended that the appropriate shear strength for use in calculating bearing capacity was the average shear strength for a depth of $\frac{2}{3}B$ below the base of the footing, provided there were no strength values exceeding ±50% of the average. However, this recommendation was based on a static footing, not one of which penetrated through sediment to reach its final depth. For mat-supported rigs, experience has shown that a depth of slightly over one-half of the minimum dimension of the mat should be used. For example, if the minimum dimension is 30 ft, the average shear strength for a depth of 15-20 ft below the base of the mat should be used, provided that the shear strength within this depth does not exceed the ±50% restriction. When the shear strength increases linearly with depth, which is often the case in normally consolidated sediments and is sometimes the case with underconsolidated sediments, Helfrich et al. (1980) found that the bearing capacity equation developed by Davis and Booker (1973), which considers a linearly increasing strength with depth, did a better job of predicting mat penetration. Furthermore, for both methods the remolded shear strength should be used rather than the undisturbed strength. For the circular footings used on the spud-can type of jack-up foundation, the significant depth equals the diameter of the footing. In both types of footings, the remolded shear strength should be used when calculating storm loading effects on the foundation.

Storms can produce large lateral loads as well as overturning loads. Lateral loads do not present a problem for the platforms supported by independent legs since the legs penetrate deep enough to resist lateral movement. However, mat-supported rigs have been known to slide along the bottom. Experience has shown that these rigs have moved laterally during storms when the shear strength of the sediment was less than 50-60 psf, but no problems have developed when the strength was greater than 100 psf. These observations have not been supported by theory or experiments to provide a rational explanation of the behavior. If the depth of penetration of the mat were unknown, which is the usual case, then it would be difficult to back-calculate the behavior.

Although the preloading operation may prevent total foundation failure during storms, the platforms often exhibit additional settling as a result of storm forces. This is termed "shakedown." Little knowledge is available for predicting the additional settlement due to shakedown. From the standpoint of rig operators, it is usually not a serious problem because shakedown settlement can be compensated for by jacking the rig up higher when the storm has passed. It would only be a problem if the resulting air gap between the deck and the water line were too small to take care of the next storm.

Shakedown may be caused by several factors. It may result from the momentary increase of bearing pressure on opposite sides of the footing as the structure rocks back and forth during the storm. Another cause may be the increase in pore pressures and resulting loss of strength due to the repeated loading of the sediment beneath the footing.

2.2.3.3 Cohesionless soils. Traditionally, the bearing capacity of sands has seldom been a problem, and this also has proven to be the case with jack-up rigs. Bearing-capacity factors recommended by Meyerhof (1971) have been used in calculation of bearing capacity with the angles of internal friction for the sediments being estimated on the basis of particle gradation. Even when the calculated bearing capacity was quite low, the resistance was much greater than the bearing pressure of the footings.

There seems to have been no recorded instances where jack-up rigs on sands have suffered sliding or skidding along the bottom due to lateral forces during storms. The platform-supported rigs usually have scour skirts which probably act to resist such movement due to the passive resistance of the sands adjacent to the skirts. Some of the larger rigs are designed to have large enough bearing pressures that sufficient penetration into the sands will occur to resist both scour and lateral movement. If there is insufficient penetration to resist lateral movement, the force required to initiate movement can be calculated on the basis of friction forces in a manner similar to that used to calculate frictional resistance of steel piles in sands. Several experiments have been performed to determine the friction coefficient between steel and sand.

In general, the problems associated with granular soils relate not to bearing capacity, but to scour effects, liquefaction, and pumping beneath the foundation under repeated loading.

2.2.3.4 Layered soils. One of the most serious problems associated with the placement of jack-up rigs has been punch-through failures where the footing initially rests on a strong soil underlain by a weaker soil. Several

geologic conditions can lead to a punch-through failure. The most serious situation occurs when the footing fails to penetrate sufficiently during setup and then punches through during application of storm wave forces. The layered soil problem has not been studied as extensively as the standard bearing capacity problem, since the usual engineering approach to such a problem would be to avoid it completely by taking the foundation element below the level of the soft material. One rather simplistic approach is to assume that the stress applied at the surface is transmitted to the underlying soil at some stress distribution ratio, say 3:1 or 4:1, as shown in Figure 2.2-8. The resulting stress at the top of the weak layer is then compared to its ultimate bearing capacity.

A few analytical methods have been developed to determine the bearing capacity of layered soils. Hanna and Meyerhof (1980) and Jacobsen et al. (1977) proposed methods for the situation of sand over clay, whereas Brown and Meyerhof (1969) developed a method for a footing resting on strong clay overlying a weak clay. Young et al. (1984) showed results of model tests that indicated that the closest fit with the data for the situation of a sand over clay was achieved by the Hanna and Meyerhof method. Their formula is

$$q_t = [6S_{ub} + 2\gamma_b H^2(1 + 2D/H)\, K_s\, (\tan \phi)/B] + \gamma_b D\,, \qquad (2.2\text{-}6)$$

where S_{ub} = undrained shear strength of the bottom soil stratum (F/L^2),

H = thickness of the upper soil stratum (L),

D = depth of embedment (L),

K_s = coefficient of punching shear (unitless),

ϕ = angle of internal friction of the sand (degrees),

γ_b = effective unit weight of the sand stratum (F/L^3).

Although there are apparently no field data to support their convictions, Young et al. recommend the Brown and Meyerhof method for the situation of a strong clay over a weak clay. The formula for this situation is

$$q_t = [3S_{ut}\,(H/B) + 6S_{ub}] + \gamma_b D\,, \qquad (2.2\text{-}7)$$

where S_{ut} = undrained shear strength of the top soil stratum and the other terms are previously defined.

2.2.4 Example Calculations. As an indication of the depth of sinkage expected with a 10-ft by 40-ft pad resting on a weak sediment, using the approaches developed in the preceding discussions, calculations were made using a sediment profile typical of those found in the near-surface sediments in the Mississippi Delta. Bearing pressures exerted by the pad of 50, 100, and 200 psf were used in the calculations, the latter values being considered as indicative of storm-induced pressures on the pad. The results are shown in Figure 2.2-9.

2.3 Breakout

2.3.1 General. Structures resting on the seafloor sediments, even for a short length of time, experience a resistance to liftoff called the "breakout" force.

On cohesionless seafloors this breakout force is due primarily to friction on the sides of the structure (Fig. 2.3-1a). If the uplift force is applied very quickly, as through the snap loading of a lift line by a floating recovery platform riding on the ocean swell, then a suction force may develop on the underside of the structure resisting liftoff. On cohesionless or sand sediments, this suction force is short-lived with the high permeability of cohesionless sediments allowing for rapid flow of pore water to relieve the suction pressures. Thus, for expected rates of uplift load application, on cohesionless sediments the structure is expected to experience breakout resistance due only to side friction forces with suction forces being negligible.

On cohesive sediments the story is quite different. The side friction force, while measurable, is generally overshadowed by the suction force on the underside of the structure (Fig. 2.3-1b). In addition to being large, the suction force is also long-lived because the permeability of a cohesive sediment is normally quite low allowing very low pore water flow rates to relieve the suction pressures resisting liftoff. Given sufficient time and appropriate sediment constituents, it is even possible for sediment-structure adhesion to develop, where the individual sediment grains are electrostatically attracted to the structure surface. However, this effect is difficult to distinguish from suction or negative pore pressure effects.

2.3.2 Analysis of the Breakout Problem: Cohesionless Soils

2.3.2.1 Problem description. Model studies of the structure resting on a cohesionless sediment show that the prototype will experience considerable lateral and rocking movement on the seafloor under expected wave and current loadings. This oscillatory lateral sliding and rocking has resulted in the excavation of a "bowl" with sand berms of a model depth equivalent to 5 ft for the prototype. If the structure were to remain in place for some time after the peak environmental loading, then it is probable that the bowl around the base would partially fill in, naturally placing sand against the sides of the foundation. This cohesionless sediment can develop a significant uplift or breakout resisting force.

2.3.2.2 Analytical approach. The sand infill against the sides of the structure is assumed to exert a normal force on the sides proportional to the effective weight of the overburden sand. This normal force acting on

the sides is responsible for a friction force between the sand infill and the sides of the structure — the friction force that is the breakout force on cohesionless seafloors.

The frictional forces of concern here arise from the effective overburden stress in the sediment infill. This effective overburden stress $(\bar{\sigma}_v)$ at depth Z is

$$\bar{\sigma}_v = \gamma_b Z , \qquad (2.3\text{-}1)$$

where γ_b = the buoyant (effective) unit weight of sediment (F/L^3). The relationship between the horizontal effective stress $(\bar{\sigma}_h)$ and the vertical or overburden effective stress is the lateral earth pressure coefficient (K):

$$K = \bar{\sigma}_h / \bar{\sigma}_v . \qquad (2.3\text{-}2)$$

Rocker (1985) gives a value of 0.5 for K when calculating the horizontal effective stress on the wall of a driven pile in sand being loaded in uplift. Kezdi (1975) quotes Meyerhof giving a value of 0.5 for K from field data in loose sand for pile capacity. Thus, K = 0.5 is assumed appropriate for the structure breakout problem in sand.

The resistance to uplift on the sides of the structure is represented as a simple Coulomb friction problem, with the friction stress, (f_f), represented by

$$f_f = \bar{\sigma}_h \tan \delta , \qquad (2.3\text{-}3)$$

where δ = surface friction angle on the side of the structure (degrees). Rocker (1985) suggests a value of $\bar{\phi}$ − 5° for the surface friction angle, where $\bar{\phi}$ is the effective angle of internal friction, for piles driven in sands. Peck et al. (1953) suggest a value of $\bar{\phi}$ = 34° for an angular-grained sand, yielding a surface friction angle of 29°, an angle which appears somewhat high for the structure breakout problem. For the breakout problem, we recommend $\delta = (2/3) \times \bar{\phi}$, yielding δ = 22.7° for the assumed effective angle of internal friction of 34°. The surface friction angle (22.7°) thus calculated corresponds well with reported surface friction angles for wet silts and sands ranging from 20° to 26° given by Kezdi (1975).

Equations 2.3-1 and 2.3-2 can be substituted into Equation 2.3-3 to yield

$$f_f = K\gamma_b Z \tan \delta . \qquad (2.3\text{-}4)$$

The friction force (not stress) (F_f) on a 1-ft width of the sidewall of the structure, with the base of the sidewall at depth D, can be shown to be

$$F_f = (2/3) \times D f_{fD} , \qquad (2.3\text{-}5)$$

where f_{fD} = the friction stress acting on the sidewall at depth D, or,

$$F_f = (2/3) \times K\gamma_b D^2 \tan \delta . \qquad (2.3\text{-}6)$$

The buoyant (effective) unit weight, (γ_b), of a loose quartz sand is given by Peck et al. (1953) as 60 pcf.

Equation 2.3-6 was used to calculate the friction force acting on a 1-ft-thick slice of a 10-ft-wide strip footing supporting the structure. The total friction force acting on the two ends of the slice is plotted in Figure 2.3-2.

The uplift pressure available to cause breakout has been given as about 9.2 psf net uplift, or, for the 10-ft slice of foundation, about 92 lb. From Figure 2.3-2, the available breakout force of 92 lb is seen to permit breakout and lift-off of the structure to embedments in sand of about 2.3 ft. (Note: A larger breakout force, larger than 92 lb per 10-ft slice, can be developed by adding buoyancy to the structure at the time recovery is initiated. However, increasing the buoyancy beyond 92 lb may cause uncontrolled ascent after breakout, and thus only complicates the problem.) Unanticipated variations in sediment parameters must be allowed for by applying a Factor of Safety (F.S.) to the ultimate available breakout force. When we are dealing with an item of high value that must not be lost, an F.S. of 3 is appropriate. Thus the allowable breakout force to be considered available is

$$F_b \ (design) = 92/3 = 31 \ lb. \qquad (2.3\text{-}7)$$

Figure 2.3-2 shows that the allowable breakout force of 31 lb is adequate to ensure breakout and liftoff for embedments up to only 1.4 ft.

2.3.2.3 Discussion. This preliminary analysis shows that a burial of the foundation of only 1.4 ft is sufficient to jeopardize recovery of the structure by uplift force alone. Model tests of the structure performance in expected environmental conditions indicate settlement of the structure of 5 ft into a plowed/scoured bowl in the seafloor sand. No record is given regarding embedment of the structure below the bottom of the bowl nor is there record of the rate of sediment infill after the abatement of maximum wave and current forces.

It appears that the structure could possibly settle by one mechanism or another on sand seafloors and then be subject to infill around its foundation to a depth of 1.4 ft. The possibility of this event occurring should be thoroughly evaluated an' if deemed to be of significance, then means should be found either to reduce the sediment friction on the foundation sides, such as by water jetting, or to increase the available uplift for breakout.

2.3.3 Analysis of the Breakout Problem: Cohesive Soils

2.3.3.1 Problem description. On cohesive sediments the structure will experience significant settlement due to immediate elastic and plastic deformation of the sediment without volume change, followed by consolidation settlement resulting from the expulsion of pore water and the movement of soil grains into a closer, tighter fabric. The magnitude of these immediate and time-dependent settlements will be on the order of several inches and may reach a few feet over the period of emplacement. These settlements have not been calculated except at one hypothetical location (Fig. 2.2-9) because (1) probable environmental conditions at proposed sites have not been made available; (2) subsequent calculations will show that the breakout force required exceeds the net uplift force available even for very small settlement values, making a refined estimate of settlement unnecessary to identify the problem—breakout of the structure on many cohesive sediments will be difficult to achieve; and (3) the magnitude and influence of current and wave forces on the structure and on the settlement of the supporting sediment are unknown, and may very well exceed the importance of static loads in this breakout problem.

Breakout from the sediments requires that either water or sediment move into the space beneath the structure as it is lifted. In the case of most cohesive sediments, the permeability is quite low and pore water cannot flow through the sediment rapidly enough to relieve the negative pore pressures developed beneath the structure by the uplift force. Thus, breakout requires that the sediment follow the structure, in a failure mechanism that is the reverse of the general bearing capacity failure. The uplift force required to deform and shear the sediment in this manner is comparable to the bearing capacity of the sediment, but reversed in direction. Breakout may be achieved with relatively smaller uplift force, but sufficient time must be allowed for the pore water flow to dissipate the negative pore pressures. The following analysis will show that the time duration required for the structure in question is too long to be a viable option.

2.3.3.2 Analytical approach to breakout force estimate. The bearing capacity of the foundation resting on the cohesive seafloor must be determined first. A sediment undrained shear strength of 0.5 psi or 72 psf is assumed herein as an intermediate value on which the structure may be sited. The bearing capacity per unit area for a structure bearing on the sediment surface is

$$q_f = S_u N_c , \qquad (2.3\text{-}8)$$

where N_c = a nondimensionalized factor relating the bearing capacity to the undrained shear strength equal to 5.14 for this case, and

S_u = undrained shear strength.

The bearing capacity on the assumed seafloor sediment is then

$$q_f = 5.14 \times 72 \text{ psf} = 370 \text{ psf}.$$

The predicted operating bearing pressure on the seafloor is 100 psf; therefore, the factor of safety in bearing for the static load is

$$F.S. = 370/100 = 3.7.$$

One must note, however, that this predicted bearing load includes only the gravity load. Current and wave loadings on the structure are significant and must be factored into the bearing capacity analysis when establishing lower strength limits for acceptable environments for siting the structure. Note that the current and wave loading dynamics may lead to partial remolding of the sediment and loss of strength and rigidity, which also must be factored into the bearing capacity analysis. A remolded shear strength may be appropriate to use when no additional dynamic strength data are available, although this may not be the minimum shear strength.

The breakout force is calculated after the technique developed by Lee (1972) and formalized in Rocker (1985). The breakout force was shown by Lee to be significantly influenced by the depth of embedment, D, of the foundation base, as shown in Figure 2.3-3. Lee's experimental work showed that for a structure resting on a cohesive seafloor, where the depth of embedment is negligible, the breakout force is very low. Because the structure is resting on the surface, slight uplift initiates a structure—sediment separation at the edges of the bearing area, the entry of water from the water column, and a prompt relief of the negative pore water pressures under the structure. Even slight embedment of the structure reduces the potential for this relief and reduction of the breakout force required.

Lee's analysis of 57 breakout tests on objects of different shape, size, and embedment depth show that the conservative breakout curve, *Curve A*, of Figure 2.3-3 should be used to ensure a confidence level of 95% on the prediction of the force required for breakout. This curve has been used to obtain the ratio of the required immediate breakout force to the bearing capacity for increasing depths of embedment of the subject structure (see Fig. 2.3-4). The ratio of available uplift force to the bearing capacity has also been presented in Figure 2.3-4 for easy comparison of available and required uplift forces. It is painfully obvious from Figure 2.3-4 that breakout of the structure from cohesive seafloors, using only the uplift force now available, will not occur immediately upon uplift load application. The embedment depth of the

structure for a range of conditions should be calculated to confirm these projections, but for this report an embedment depth of 1 ft is estimated to require an uplift force five times that available for breakout.

2.3.3.3 Time to breakout. Breakout may be achieved at lower uplift loads than required for immediate breakout by applying those lower loads for a long period of time, thus allowing for partial dissipation of negative pore pressures and sediment creep to assist in achieving breakout. Presently, no theoretical solution for determining breakout time of flat-bottomed footings is possible. From his limited data, Lee has offered an empirical relationship for predicting time to breakout (Fig. 2.3-5). Both axes of Figure 2.3-5 are normalized with the ordinate being the ratio of the applied uplift or breakout loads (F_{lb}) to the immediate breakout load (F_{ib}). The abscissa of this figure is a breakout time parameter, T.

The time to breakout for a given relative breakout force can be calculated from

$$t_b = TD^4/B^2p \,, \tag{2.3-9}$$

where p = average breakout pressure applied to the sediment in psf,

t_b = time to breakout in minutes, and

D and B = embedment and width in feet.

Curve A of Figure 2.3-5 was used to estimate the time to breakout of the object subject to the available uplift pressure of 9.2 psf for depths of base embedment of 0, 1, 2, 3, and 4 ft. The time to breakout for these embedment depths are as follows:

Base Depth (ft)	F_{lb}/F_{ib}	t_b (hours)
0	1.32	0
1	0.18	3
2	0.10	1,000
3	0.08	22,000
4	0.07	140,000

These time-to-breakout predictions are considered highly unreliable by the authors; however even making allowance for an order-of-magnitude error, these predictions indicate that we cannot rely on long-term breakout as a means of recovering the structure from cohesive seafloors - the long time durations required are unacceptable (and probably too unreliable!).

2.3.3.4 Aids to breakout. Because the prospects of structure breakout from cohesive sediments appears tenuous at best from the above calculations, it is necessary to consider possible methods for aiding breakout. Potential aids are water jetting, eccentric application of the breakout force, and electro-osmosis.

Lee has discussed the application of water jetting through perforated tubes along the sediment-structure contact surface. No guidelines exist for the design of these tubes, other than that spacing of the water jet openings should be minimized. Field tests have shown that a 50% to 77% reduction of the required breakout force is possible with the use of water jets.

A reduction in the required breakout force also will result if the uplift force is applied at one end of the structure rather than through its center. The use of an eccentric uplift force may reduce the magnitude of the breakout load required by up to 50%, with the technique demonstrating the most benefit when applied to long, narrow structures.

Electro-osmosis involves the application of a direct electric current to the sediment below the structure so that the pore water will flow toward the negative electrode. The skin of the foundation of the structure would serve as the negative electrode, while several probes embedded below the structure would serve as the positive electrodes. This concept is used for dewatering sites of building foundation excavations and for lubricating knives used to trim sediment samples in the laboratory, and has been proposed to the Supervisor of Salvage for reducing the force required to free vessels grounded on mud banks. Before trying to apply this concept to the problem at hand, some preliminary work is needed to verify that electro-osmosis will work in a significant number of the cohesive sediment types to be encountered by this structure. (Electro-osmosis works well in many, but not all, terrestrial cohesive soils.)

2.3.3.5 Discussion. This preliminary analysis of breakout force required on cohesive sediments reveals that breakout of the structure from clays will be very difficult if any settlement of the object occurs. Certain parts of this analysis should be refined in a second cut; however, the inadequacy of the available breakout force suggested by the preliminary analysis is so large that refinement of the analysis is not expected to improve the overall picture—that is, we may not be able to break the structure free of a soft cohesive seafloor without a significant increase in the available breakout force.

2.3.4 Complication of Breakout Problem by Sediment Accumulation

2.3.4.1 Cohesionless. The foundation of the structure is not watertight, and sand has been found inside the foundation. The sediment inside the foundation has a unit weight higher than that of the seawater it displaces; the unit weight of a loose, mixed grained sand is reported as 124 pcf (Peck et al., 1953), giving a resulting buoyant unit weight of 60 pcf. The net uplift pressure available for breakout is only 9.2 psf. This 9.2 psf is equivalent to the additional gravity load or negative buoyancy of 1.8 inches of loose sand distributed uniformly over the inside of the foundation. Thus, less than 2 inches of sand inside the foundation can effectively prevent recovery of the structure.

This sand apparently works its way into the foundation structure due to a pumping action caused by the oscillating water flow around the foundation due to waves and due to oscillating movement of the foundation on/in the sand due to wave forces. It appears imperative that the foundation element be sealed wherever the entry of sand is possible.

2.3.4.2 Cohesive. Cohesive sediments should be expected to adhere to the foundation of the structure and to come away from the seafloor adhered to the sides and base of the foundation. The firmness and thickness of this adhered layer or skin should be expected to increase with time. No data collection on this phenomenon and no predictive technique on the skin thickness are known. This adhered skin is, however, a potential problem to breakout: the cohesive sediment adhered to the foundation base will probably have a buoyant unit weight of approximately 30 pcf, and a 3.3-inch skin of this material is sufficient to balance the available uplift force and prevent recovery.

2.3.5 Conclusions. This very brief review of the breakout phenomenon as it occurs for a particular structure indicates that a very critical problem exists. On cohesionless seafloors, if the structure should settle by scour or other mechanism, and then if sand should be filled against the side of the foundation to a height of 2.4 ft, the available breakout force would be inadequate to overcome the friction force of the sand against the foundation sides and the structure would not break free of the seafloor. On many cohesive seafloors, even very small settlements of a few inches will result in development of significant resistance to breakout, far exceeding the available breakout force.

These conclusions are based on a limited review of the breakout problem. The problem requires reexamination with additional and improved input data and more refined evaluation of some facets of the problem before preliminary operating guidelines can be prepared for siting the structure.

2.4 Resistance to Lateral Movements

2.4.1 General. Waves and currents acting on a bottom-resting structure produce forces leading to lateral movement. The extent of lateral movement for a given forcing function depends on the type of sediment and also on the amount of penetration of the structure into the seafloor. The resistance to lateral movement is governed by the friction between the base of the structure and the sediment (cohesionless sediments), or by the adhesion between the base and the sediment (cohesive sediments). If the structure penetrates into the seafloor, passive resistance of the sediment against the sides or walls of the structure must be considered. This component is often neglected in conventional marine geotechnical designs, since it is small compared to the base resistance. For our problem, the reverse may be true, as will be shown later.

Model tests on the structure indicated that lateral movement was an important aspect of the structure behavior under the action of waves. The oscillatory nature of wave forces can result in back and forth movement of the structure on the seafloor, which can be detrimental to the mission of the object. The purpose of this section is to discuss those factors that govern the resistance to lateral movement. Since the mechanisms involved are different for cohesionless and cohesive sediments, the following discussion considers the sediment types separately.

2.4.2 Lateral Resistance–Cohesionless Soils. The lateral resistance in cohesionless sediments can be expressed as

$$Q_l = W \tan \delta + \frac{K\gamma_b H^2 L}{2}, \qquad (2.4\text{-}1)$$

where Q_l = total lateral resistance (force),

W = total vertical force applied by the footing,

H = the depth of footing embedment into the sediment,

γ_b = the submerged unit weight of the sediment,

δ = the friction angle between the sediment and the footing,

L = the contact length of the footing (leading edge),

K = earth pressure coefficient.

The friction angle, δ, between a footing and the sediment has been obtained in various studies relating to prediction of bearing capacity of piles and earth pressures on retaining walls and bulkheads. Values commonly used in design range from $2/3\bar{\phi}$ to $\bar{\phi}$ -5°, where $\bar{\phi}$ is the effective angle of internal friction of the sediment.

The earth pressure coefficient, K, for passive pressures depends on both $\bar{\phi}$ and δ. It can be obtained for simple loading conditions from such charts as Figure 2.4-1, which assumes a linear failure surface. For our problem, this assumption will lead to a 20% to 30% underprediction of the lateral force.

Figure 2.4-2 shows the results of lateral resistance calculations for a 10-ft by 40-ft footing at different depths of penetration into the seafloor, assuming two different bearing pressures (50 and 100 psf), with lateral force oriented at two directions to the axis of the footing. Additional parameters are shown on the figure. Figure 2.4-1 was used to obtain K.

The figure shows that the lateral resistance is relatively small if the footing is not embedded in the sediment. This is the result of the very light bearing pressures imposed by the footing. The majority of the lateral resistance comes from the passive resistance of the sediment along the leading edge of the footing.

This means that if the structure is located in a basin created by some other means, such as scour, the footing could easily slide under imposed lateral loads until it contacted the sidewalls of the basin, at which time it could come to an abrupt stop. It can also be observed that the lateral resistance is significantly influenced by the direction of movement (A or B directions in the figure).

The approach used does not consider dynamic effects, nor does it consider the reduction of the lateral resistance beneath the footing that might occur with repeated sediment loading and pore pressure generation. The former would increase the lateral resistance, whereas the latter would reduce it. If berms are formed at the lip of the basin, the lateral resistance would be increased.

2.4.3 Lateral Resistance-Cohesive Soils. In cohesive soils, the lateral resistance can be obtained for two conditions: long term and short term. Only the latter is considered appropriate for the problem at hand. For the short-term condition, adhesion of the sediment to the base must be considered. This is independent of the normal force applied to the base. The formula for determining the short-term lateral resistance is:

$$Q_l = (A_s)\, a + 2\, S_u A_c\,, \qquad (2.4\text{-}2)$$

where A_s = the area of the base in contact with the sediment, a = the adhesion of the sediment, A_c = contact area of the leading edge of the footing. When the footing has penetrated into the sediment, adhesion should also be considered for the contact area along the sides and trailing edge of the footing.

Classically, the adhesion force per unit contact area is considered to be the undrained shear strength of the sediment, i.e., $a = S_u$. This approach is used when calculating bearing capacity of piles in cohesive soils. There are also indications in pile capacity calculations that there is a limiting value of adhesion: 1000 psf is often used. Whether a limiting value is also the case for adhesion such as considered here is not known.

Use of the previous formula in clays has several shortcomings. First, when lightly loaded footings rest on stiff clays (which do exist at the seafloor in some marine environments) the contact area between footing and clay will likely be very low unless the clay surface is very smooth. Calculations using the full contact area, A_s, rather than the actual but unknown contact area, are likely to significantly overpredict the lateral resistance. Also, with a stiff clay, a stick-slip mechanism may occur such that reduced sliding resistance occurs after initial breakaway. Second, very soft clays (muds) will not behave in the manner assumed in the development of the formula. As will be discussed later, soft clays behave as viscoelastic materials, which means that their shear resistance (and adhesion) is a function of the loading or strain rate. Recent research conducted on the resistance of buried and partially buried pipelines to sediment movement has shown the importance of velocity on resistance to movement of these pipelines (Schapery and Dunlap, 1984).

The pipeline research also showed that a complex pattern of sediment cracking near the pipeline and separation at the pipe—sediment interface occurred under certain conditions. The cracking and separation significantly influenced the resistance to lateral movement of the pipeline. An increase in the total pressure on the sediment, which is analogous to an increase in water depth, suppressed the cracking and separation and effectively changed the resistance to lateral movement.

It was mentioned earlier in the section on bearing capacity that no platform-type jack-up rigs in the Gulf of Mexico had suffered lateral movement during storms if the supporting sediment had a shear strength greater than 50 psf. It would be interesting to calculate the lateral forces on these rigs and determine if the actual behavior could be predicted using the presently available theories.

2.4.4 Effective Depth of Soil Properties. The effective depth to which the sediment properties govern the lateral movement is not well recognized. For cohesionless materials, it is felt that the depth of sediment involved may be only a few inches below the base of the footing. This means that the angle of internal friction and base friction are needed only to the approximate depth to which the footing will eventually penetrate. The research conducted on marine pipelines in soft sediments showed that the sediment strength to a depth of one-half the pipe diameter below the bottom of the pipe was important.

2.5 Alternative Approach in Sinkage and Breakout Calculations

Problems related to pipeline movement in soft deltaic sediments were recently investigated at TAMU for the American Gas Association (Schapery and Dunlap, 1984). Although this research was primarily related to lateral forces exerted by sediment moving against pipelines, both the theoretical and experimental aspects considered the forces required to push a pipe into the sediment and extract a buried pipe from the sediment. Even though the configuration of a pipe is different from that of a flat footing, the general principles involved are the same.

Based on the fact that soft marine sediments behave as nonlinear viscoelastic materials, the forces required to push or pull (breakout) a structure from soft sediment should be a function of velocity of movement.

Drag tests performed on model pipelines confirmed this and showed that the rate effects could be accounted for by the use of a term which included the pipe velocity and the viscoelastic rate exponent. The total vertical force per unit length of pipe was found to be

$$F = DS_u(V/D)^n g_2 + F_B , \qquad (2.5\text{-}1)$$

where D = pipeline diameter,

S_u = undrained shear strength of the sediment,

V = relative velocity between the pipeline and the sediment in the free field,

n = viscoelastic rate exponent for the sediment,

F_B = buoyant force per unit length of pipe,

g_2 = force coefficient.

Values of g_2 were obtained both experimentally and theoretically for different initial burial depths of the pipe. The experimental results are shown in Figures 2.5-1 (pipe pushed down) and 2.5-2 (pipe pulled out). Burial depths are expressed as h/D, where h is the distance from the bottom of the pipe to the sediment surface (for example h/D = 0.5 indicates the pipe is half-buried). The force coefficient, g_2, is analogous to the bearing capacity factor N_c, first used in Section 2.2 and shown in Figure 2.2-6, but it does not have the same numerical values as N_c, owing to the geometrical differences between a rounded pipe and a flat bearing surface.

A point which shows up in these results is that the values of g_2 are not the same for push down and pull up. For example, the maximum g_2 for a half-buried pipe being pulled out is -3.0, which actually occurs when the pipe reaches a burial depth during extraction of h/D = 0.4. For a pushed down pipe at h/D of 0.4, the value of g_2 is 6.5 (the difference in signs reflects the difference in direction of forces between push down and pull out).

The ratio of force coefficients for the pull-up versus push-down case is 3.0/6.5 = 0.46, which indicates that roughly half the pushing force is required to pull the pipe out (breakout) of the sediment for this particular burial depth. This can be compared to the ratio of forces shown in Figure 2.3-3 for immediate breakout of flat-bottom footings. The correlation cannot be exact because of the different geometries, but using the "best estimate" curve in Figure 2.3-3 for an embedment depth, D/B, of 0.4, the ratio of forces is seen to be approximately 0.7, as opposed to 0.46 for the pipe case. The relative closeness of these two numbers—obtained at different agencies and at different times—is encouraging even though a direct comparison of pullout or breakout forces cannot be made. It is possible that the comparison would be even better if rate effects were incorporated into the results given in Figure 2.3-3.

It is believed that this would be a fruitful area for additional study, since the incorporation of rate effects could have a significant influence on predictions of breakout forces. As an example, the viscoelastic constant, n, is approximately 0.1 for a typical Mississippi Delta sediment. Using this n value, an order of magnitude decrease from a standard breakout velocity would result in a 25% decrease in breakout force in these sediments, or an order of magnitude increase in breakout velocity would result in a 25% increase in breakout force.

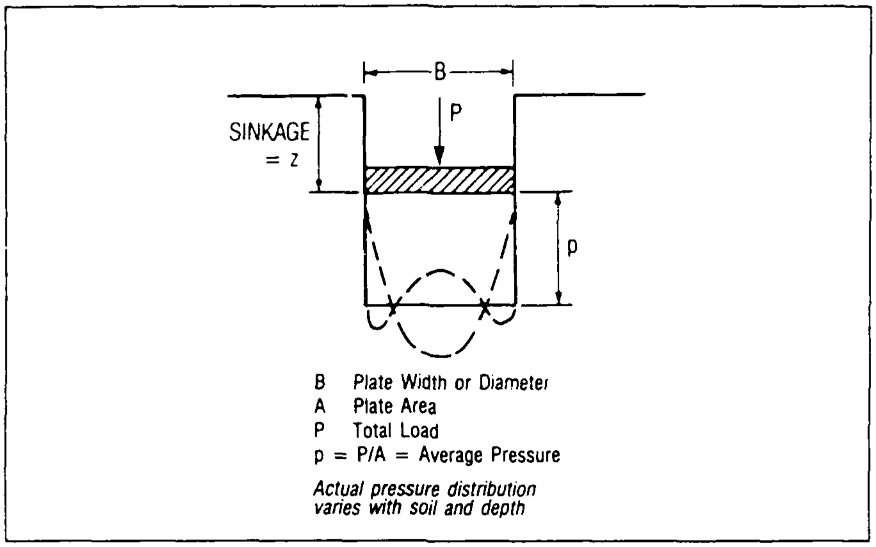


Figure 2.2-1. Plate sinkage test (after Hvorslev, 1970).

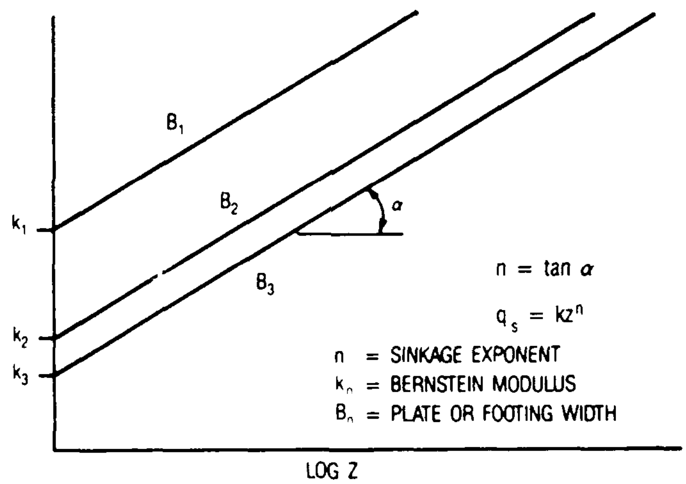


Figure 2.2-2. Plate sinkage diagram (after Hvorslev, 1970).

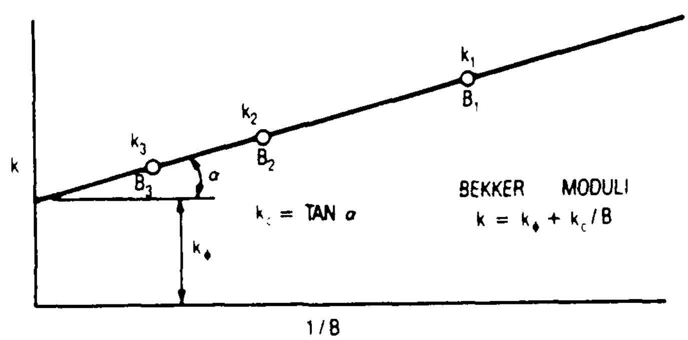


Figure 2.2-3. Sinkage moduli diagram (after Hvorslev, 1970).

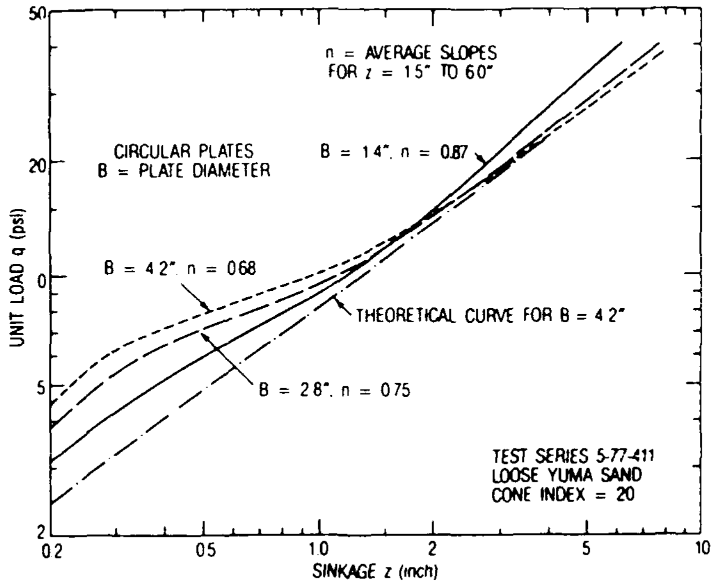


Figure 2.2-4. Sinkage test results on dense sand (after Hvorslev, 1970).

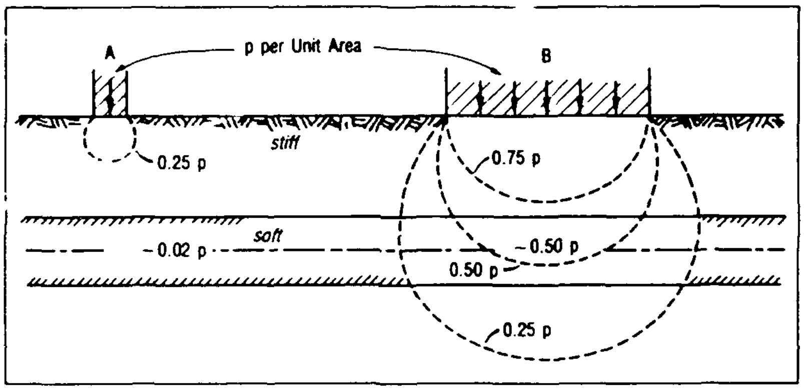
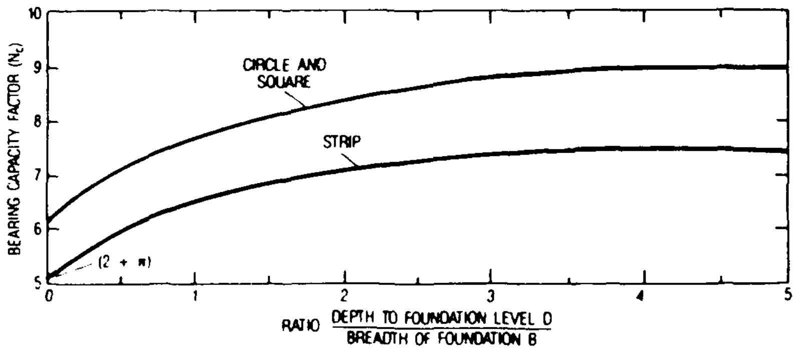


Figure 2.2-5. Significant stresses induced by model and prototype footings (modified from Terzaghi and Peck, 1967).



D/B	N_c CIRCLE	N_c STRIP
0	6 2	5 14
0 25	6 7	5 6
0 5	7 1	5 9
0 75	7 4	6 2
1 0	7 7	6 4
1 5	8 1	6 8
2 0	8 4	7 0
2 5	8 6	7 2
3 0	8 8	7 4
4 0	9 0	7 5

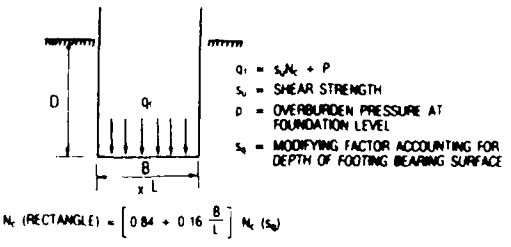


Figure 2.2-6. Bearing-capacity factors for foundations in clay (ϕ = 0) (after Skempton, 1951).

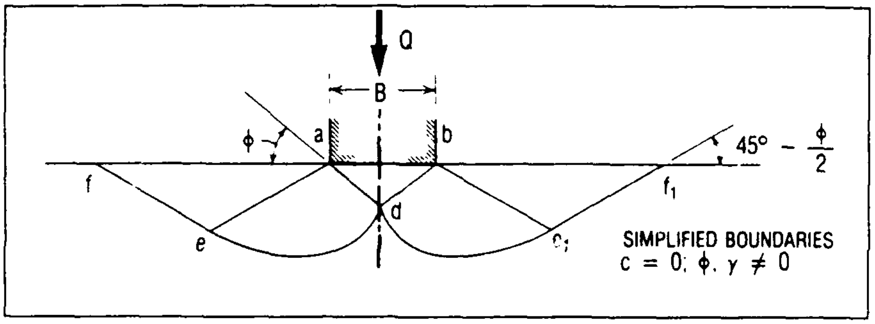


Figure 2.2-7. Failure surface assumed in bearing capacity formulas (modified from Terzaghi and Peck, 1967).

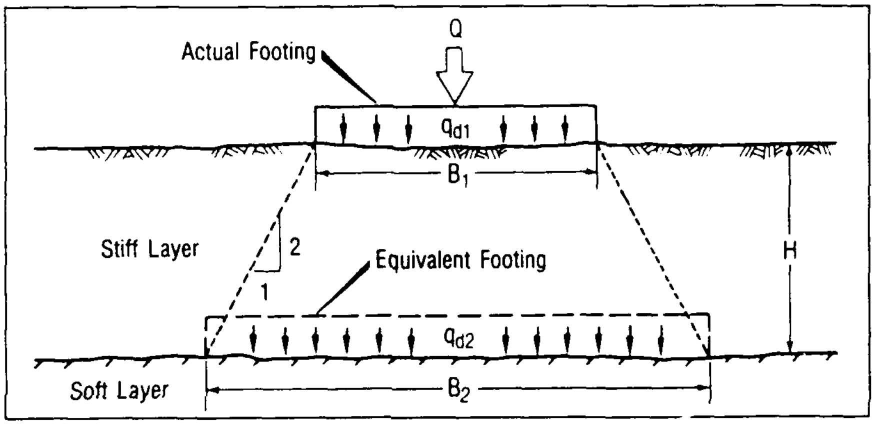


Figure 2.2-8. Simplified approach to the bearing capacity of layered soils (after Perloff, 1975).

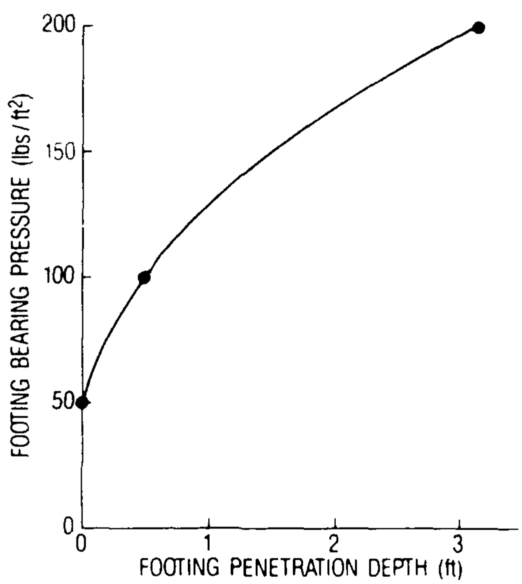


Figure 2.2-9. Estimated penetration for a 10 ft x 40 ft footing into typical delta sediment.

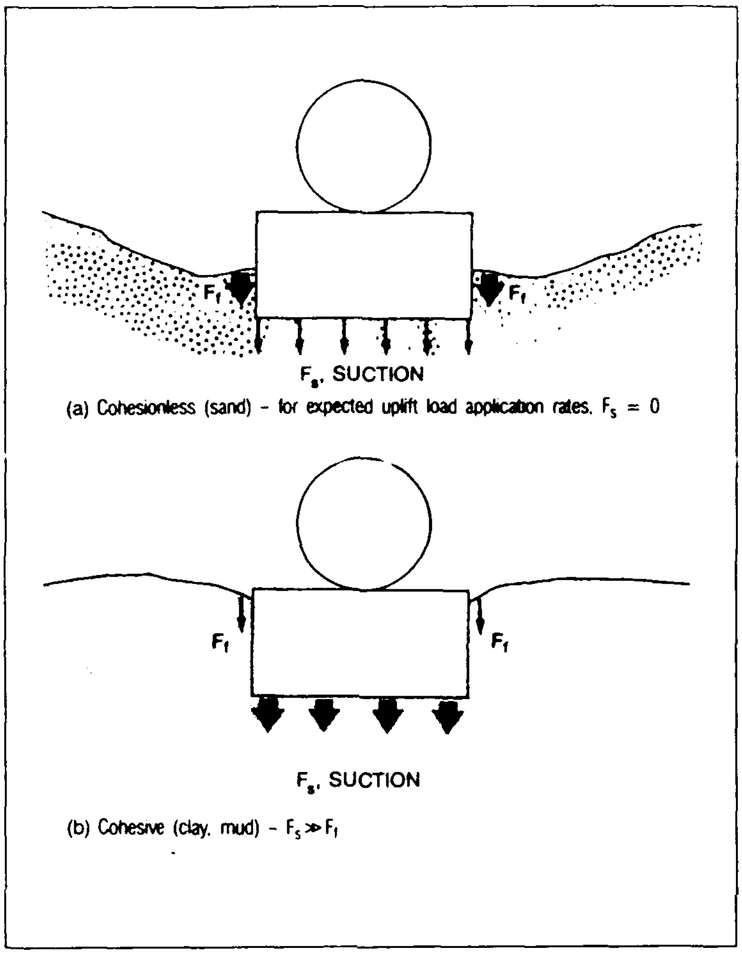


Figure 2.3-1. Comparison of breakout force components on sands versus clays.

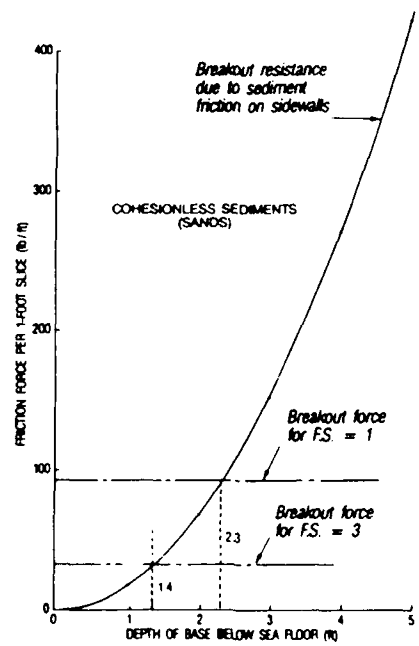


Figure 2.3-2. Depth of base below seafloor (ft). Breakout resistance of 1-ft slice of 10-ft-wide foundation in sand as function of embedment depth.

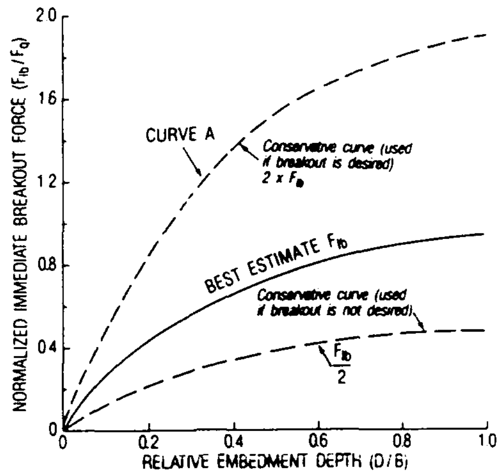


Figure 2.3-3. Normalized immediate breakout force as a function of relative embedment depth (after Rocker, 1985) [F_{Ib} = immediate breakout force carried by the sediment (F); F_q = bearing capacity of the sediment under a vertical downward load (F)].

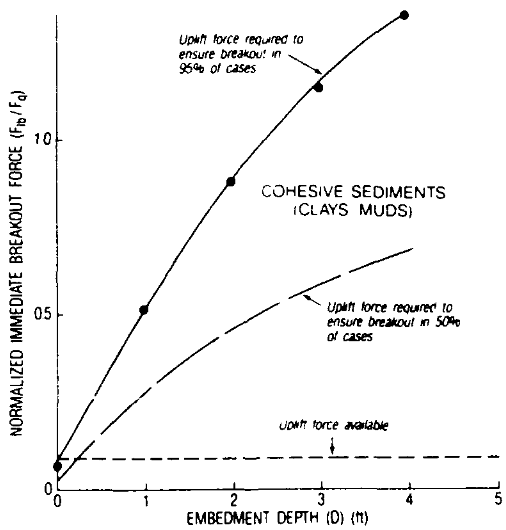


Figure 2.3-4. Breakout resistance of 1-ft slice of 10-ft-wide foundation in clays and muds as function of embedment depth.

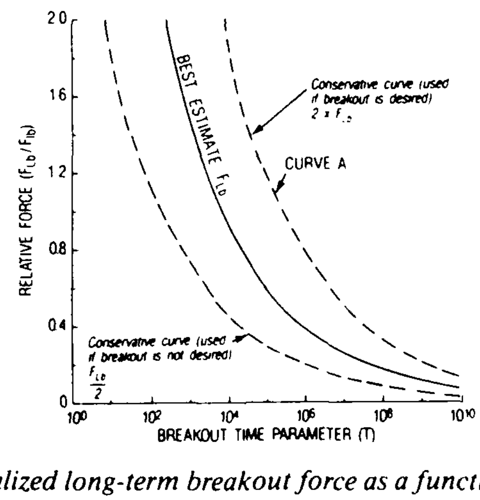


Figure 2.3-5. Normalized long-term breakout force as a function of breakout time parameter (after Rocker, 1985) [F_{LB} = long-term breakout force carried by the sediment (F); F_{Ib} = immediate breakout force carried by the sediment (F)].

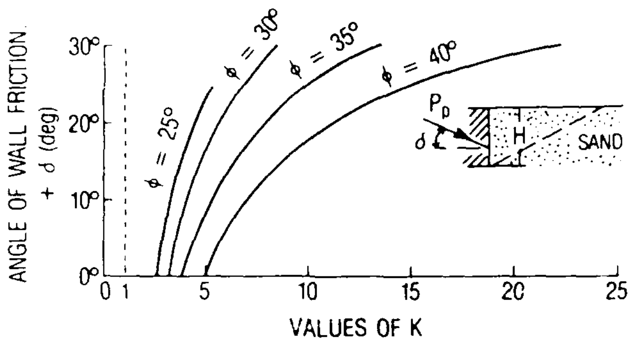


Figure 2.4-1. Chart for obtaining passive earth pressure coefficient (from Terzaghi and Peck, 1967).

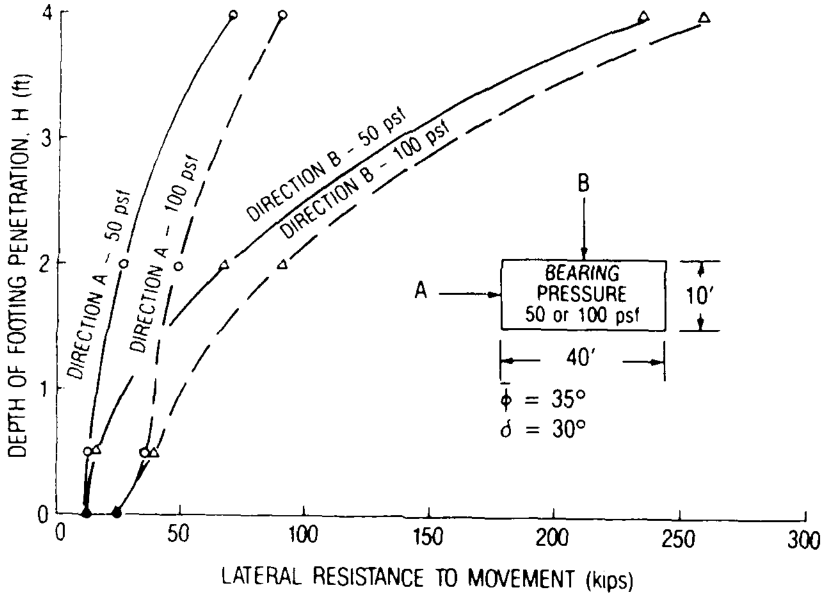


Figure 2.4-2. Maximum lateral resistance for footing in sand.

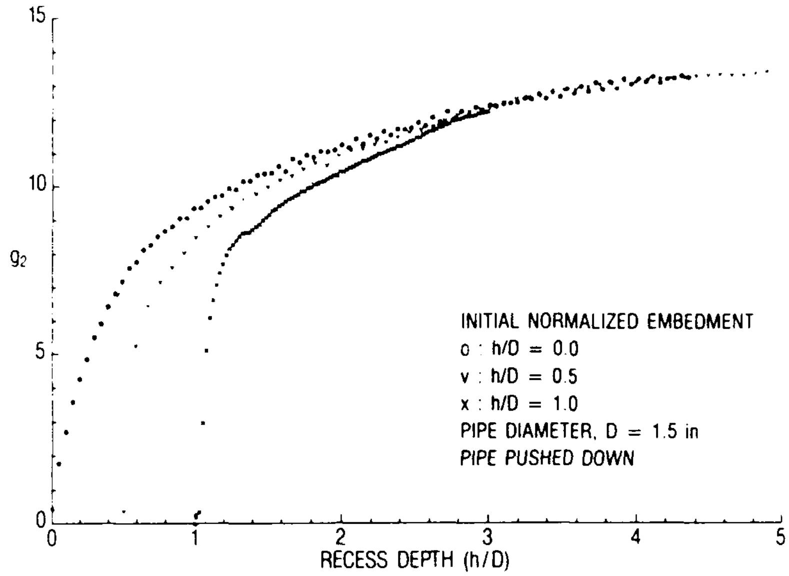


Figure 2.5-1. Vertical force coefficient for push down tests (modified from Schapery and Dunlap, 1984).

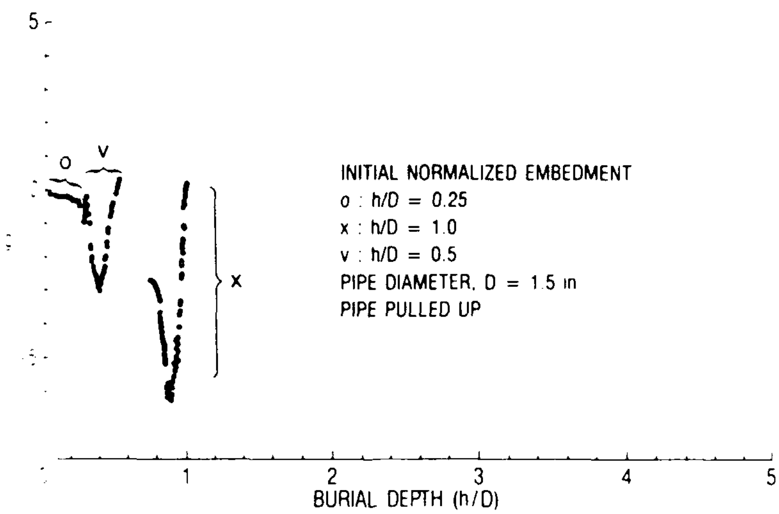


Figure 2.5-2. Vertical force coefficient for pull-up tests (modified from Schapery and Dunlap, 1984).

3.0 Mass Sediment Movements

3.1 General

As mentioned in section 1, several types of mass movements can occur in ocean sediments and these can adversely affect a structure resting on the bottom. In some cases, submarine slides result from oversteepened slopes. These failures occur in deltas and estuaries where the sediments reach a critical slope angle owing to sediment deposition. The heads of submarine canyons are particularly susceptible to slope failures because of steeper slope inclinations and their action as sediment traps. Other slope failures occur as a result of external means, including current erosion at the toe of the slope, earthquake tremors, and nearby construction activities. An important mass sediment movement, only recently understood, results from bottom pressure changes caused by storm waves.

Any mass sediment movement is likely to cause adverse effects on a structure resting on or embedded in the bottom, including total or partial burial. The presence of a structure on the bottom is not likely to trigger such movements. In some cases, earthquake-induced movements for example, there is no way of foreseeing when such movements will occur, and it can only be determined that the stage is set for these movements due to slope angles and sediment strengths. In other situations, mass movements occur with such regularity that predictions of movement can be made with considerable accuracy. The submarine cables near the mouth of the Magdelena River break most frequently in August and from late November to early December. These breaks are probably due to submarine slides, and the periods of the frequent slides correspond to times when the river has just deposited its greatest sediment load. The major recorded sediment movements in the Mississippi Delta occur during periods of intense storm wave activity, either during the winter or during the hurricane season.

3.2 Wave-Seabottom Interaction

3.2.1 The Nature of Submarine Slides. There is abundant evidence in the technical literature of seafloor movements, both ancient and modern (cf., Morgenstern, 1967). Perhaps the first detailed study of submarine slope instability was reported by Terzaghi (1956), who applied conventional static slope stability methods to explain the formation of "valleys" within the shallow waters of the Mississippi Delta front. This confirmed earlier thoughts by Shepard (1955) that the valleys were the result of downslope mass movement. Subsequent to these studies, other mass movements in the Mississippi Delta front sediments were recorded (Coleman et al., 1978), as well as submarine movements at other locations around the world.

Although some offshore platform designers had been aware for many years of submarine movements in the Mississippi Delta sediments, the damage to three platforms in the South Pass lease area during Hurricane Camille in 1969 initiated studies on these movements. After Camille, one production platform in 325 ft of water was found on its side, and two nearby platforms were so severely damaged that they had to be removed. The damage could not be explained by wind and wave forces alone. Furthermore, the uprooted platform was located downslope from the original location, although the wind and wave forces should have moved it in an upslope position (Bea, 1971).

Henkel (1970) was the first to show the direct influence of storm waves on sediment instability. His approach utilized the bottom pressures calculated for a sinusoidal water wave (Fig. 3.2-1), which acted as a pressure couple or disturbing moment on a circular arc failure plane. Beneath the crest of the wave there is a pressure increase, $+\Delta p$, and beneath the wave trough, there is a pressure decrease, $-\Delta p$. A total stress approach was utilized, and the results were presented in terms of a constant ratio of sediment undrained shear strength to effective stress below the mudline, i.e., $S_u/\gamma'_b h$. Inertial forces of the moving sediment were not considered. Henkel also warned that the damping of the wave owing to work done against the shearing strength of the soil could be an important factor that was not considered in his analysis.

For the more common situation where the shear strength varies nonlinearly with depth, it is not a difficult matter to apply the design wave pressures to the mudline and determine, by the usual trial and error method of slope stability analysis, that circle which has the minimum factor of safety (Fig. 3.2-2).

It is obvious that as storm waves pass a given point, the soil will oscillate back and forth in response to the bottom pressures, and it is difficult to visualize how a single failure circle—even one which moves in response to the waves—can adequately model the real situation. Nevertheless, this approach seems to be useful as a first approximation of whether there is a danger of submarine slides at a given location. (To our knowledge, studies comparing Henkel's approach with the more recent constitutive approaches have not been reported.) The oscillatory movement of the sediment in response to the transient pressure change can result in a buildup of pore pressures in the sediment with a resulting decrease in strength. Esrig et al. (1975) describe laboratory testing and analysis methods that can be useful in determining the strength decrease.

3.2.2 Slope Stability Methods Using Constitutive Relations. Mass marine movements in most cases do not extend far below the mudline. Since the foundations for most marine structures must extend to significant depths for bearing capacity purposes, the foundation elements usually go well below the depth of movement. However, the foundations must be strong enough to withstand the additional forces

imposed by the movement of the sediment. It is generally recognized that the more rapid the sediment movements, the greater the force exerted on the foundation. Thus, the more recent efforts aimed at solving the sediment instability problem have been directed toward determining the actual amount of movement, as well as the velocity of movement. Methods used are discussed below.

3.2.2.1 Soil modeled as an elastic material. The initial effort at solving the bottom stability problem under wave-induced loading, where the classical slope stability methods were not used, appears to have been accomplished by Wright and Dunham (1972). They modeled the sediment as a nonlinear elastic material and utilized finite element methods to calculate sediment deformations resulting from storm wave bottom pressures. Bea and Arnold (1973) utilized this procedure to evaluate sediment movements in South Pass, Block 70. Subsequently, Wright (1976) modified this approach, and only the latter method will be discussed here.

The stress-strain properties utilized in this approach can be obtained from a triaxial compression test. Relationships for both loading and unloading (stress increase and stress decrease) are needed. The loading stress-strain relationship is defined as a hyperbola as follows:

$$\sigma_1 - \sigma_3 = E_i \frac{\varepsilon}{1 + \dfrac{E_i}{2S_u} - \dfrac{1}{\varepsilon_f}} , \qquad (3.2\text{-}1)$$

where $\sigma_1 - \sigma_3$ = principal stress difference,
ε = axial strain,
S_u = undrained shear strength,
ε_f = axial strain at failure,
E_i = initial slope of the stress-strain curve.

The hyperbolic stress-strain curve is shown in Figure 3.2-3. For unloading, the stress-strain curve is assumed to be linearly elastic.

The program considers gravity stresses and a sloping subbottom. As such, the upslope and downslope lateral movements due to sinusoidal waves are not the same. However, the program does not directly provide information concerning any permanent or accumulative downslope movements. Wright (1976) discusses a method that can be used to determine these movements, although at present, it involves additional laboratory testing.

Cyclic loading effects can also be considered by taking into account the reduced values of shear strength and modulus obtained on laboratory repetitively loaded specimens. Figure 3.2-4 is an example of the influence of cyclic degradation. This example (Wright, 1976) is based on a 1000-ft-long, 70-ft-high wave acting in a water depth of 325 ft (bottom pressures calculated from linear wave theory) with the subbottom at a slope of 0.5%. The shear strength values were those reported by Bea and Arnold (1973) at South Pass, Block 70. Note that for the first loading cycle, the maximum downslope movement is approximately twice the upslope movement for this particular slope angle. The degradation resulting from 10 load cycles changes the upslope movement only slightly, but the downslope movement is nearly tripled.

It should be noted that this program does not consider the influence of bottom movement on the wave characteristics.

3.2.2.2 Soil modeled as a viscoelastic material. This method was developed for Chevron, Gulf, and Mobil Oil Companies at TAMU. The general concept was reported by Schapery and Dunlap (1978). It is based on the results of a preliminary examination, which indicated that very soft sediments behaved more as a viscoelastic rather than an elastic material. Viscoelastic materials deform with time under a constant load, their stiffness increases with the rate of loading, and the stress-strain curve for unloading is different than for loading. Since the process of testing soft samples that could hardly stand under their own weight and that had been severely disturbed by degassing during retrieval from the seafloor seemed to be undesirable, the physical testing was based on in situ testing considerations. At the time, the only device being used with any regularity as a downhole strength-measuring device was the in situ vane.

With this in mind, a miniature laboratory vane shear device was constructed for use on core samples obtained from the sediment (Stevenson, 1974). The four-bladed laboratory vane is a scale model of the in situ device now used. It has a transducer to measure the torque, T, required to shear the sample, an angular position transducer to measure the vane rotation, θ, and a drive mechanism capable of accurately controlling the rate of rotation of the vane. The device is lowered into the sample and torque versus rotation data are obtained. This is repeated at different positions in the sample at a different rotation rate until at least four different rates have been used. Typical test results are illustrated in Figure 3.2-5.

A particular rotation angle, say θ_a, is selected and the viscoelastic secant shear stiffness, $\bar{G}$, is calculated for each rotation rate by:

$$\bar{G} = \frac{T_n}{\theta_a} , \qquad (3.2\text{-}2)$$

where T_n is the normalized torque obtained by dividing the actual torque by a parameter involving vane dimensions.

At very small angles, knowing the actual time for each rate of rotation, a plot of $\bar{G}$ vs. time on log-log scales is obtained (Fig. 3.2-6). The process is then repeated for new angles. The results show that the material follows a power law in time:

$$\bar{G} = \bar{G}_l t^{n_o} \qquad (3.2\text{-}3)$$

The slope of the $\bar{G}$-t lines is n_o, and their intercept at t = 1 sec is $\bar{G}_l$. These two variables define the viscoelastic characteristics needed in the dynamic analysis. The nonlinearity of the sediment is seen from the fact that $\bar{G}_l$ varies significantly with the angle of rotation.

Limitations in operation of the in situ vane have prevented similar experimentation in situ. However, it is possible to use viscoelastic properties obtained on laboratory core samples to predict the torque for comparison with in situ torque versus rotation data. The in situ data were obtained from bore holes immediately adjacent to and at the same depth as the core samples. The results (Fig. 3.2-7) indicate that the difference between laboratory and in situ data is not large. However, laboratory data are now used only to find the value of n_o and the ratio of shear stiffness to strength. The values of in situ strengths are then used to calculate shear stiffness.

Additional theoretical analysis was needed before $\bar{G}_l$ and n_o could be used in the dynamic analysis. The vane rotation angle is related to the shear strain in the sample at the outer edge of the vane. This required a detailed deformation analysis of the vane test specimen, which was simplified by assuming that the mud between the vane blades rotated rigidly with the vane. Later, this was verified experimentally by King (1976) using a roughened circular tube, now termed the "cylindrical vane." The cylindrical vane also has the advantage that samples can be tested under confining pressures approximating the in situ pressures, and cyclic loading can be accomplished.

Using the above information, the effective viscoelastic modulus for cyclic loading can be calculated, which is the "complex modulus," a function of wave period and strain amplitude. The extension of this information to a realistic three-dimensional state of stress is accomplished by means of nonlinear viscoelastic constitutive theory (Schapery, 1968; 1974).

In predicting the dynamic response of the sea bottom to the action of storm waves, the effect of sediment inertia and gravity on the waves must be considered. This is necessary because the water waves are quite long in some cases (often >1200 ft) and the sediment modulus is very low. Because of the large hurricane wave heights, the clay response is strongly nonlinear. In addition to the nonlinear behavior, the problem is complicated by the fact that the material response depends on the strain rate. The problem is solved by first assuming the clay to be linearly viscoelastic and then the process of equivalent linearization is employed to relate the linear viscoelastic properties to the actual nonlinear viscoelastic properties. This is basically a computerized analytical solution to the governing equations and is not a finite element or finite difference type approach. The input data required are the appropriate values of n_o, in situ vane shear strength, and unit weight for up to 10 sediment layers. The water depth and parameters for a sinusoidal water wave (period, wave length, and height) are also required. The computer program output provides strain and displacement of the sediment at various depths throughout the sediment and at various intervals as the wave passes over the sediment.

Figure 3.2-8 is an example of the type of information produced by the theory and the computer program. The left side of the figure shows the idealized shear strength profile with depth. In this case nine layers of varying strength are used. A crust of higher shear strength is shown to exist from 30 ft to 50 ft and two cutback zones of lower strength are shown centered at about 60 ft and 78 ft. The right side of the figure shows the maximum cyclic displacement of the soil as a result of a 50-ft wave acting on the sediment in a water depth of 200 ft. The bottom slope in this case is 0.5%. Figure 3.2-8 shows the effect of varying the wave period by 1-sec intervals, which also influences the wave length, λ. As the wave length increases, the cyclic soil displacement increases significantly. It is noted that there is a displacement gradient or cutback in the displacement at the depth of the lower zone of weaker shear strength, which is a direct result of the soft layer at this level. Whether or not such a cutback in displacement occurs is a function of the presence of such a soft layer and other parameters, including the depth of the soft layer and the length of the water wave. The sediment above the soft layer not only oscillates back and forth; if the mudline is sloped, then it also moves downslope. If the amount of oscillatory movement increases, then the rate of downslope movement also increases. Figure 3.2-9 shows how the horizontal displacement is influenced by changing the strength of the weak layer. In this example, the bottom is sloped at 0.2%, and a 50-ft, 9.5-sec-period wave is applied. The shear strength of the weak layer is varied incrementally from 60 psf to 10 psf. As the shear strength is reduced in the weak layer, the oscillatory movement increases and the downslope velocity also increases. Of course, the decrease in shear strength in such a slope could progressively occur as a result of pore pressure buildup from repeated stressing.

The program also has two other advantages. First, it predicts the phase relationship between the forcing function (the water waves) and the sediment. This relationship is rather complicated. However, the extreme situations are shown in Figure 3.2-10. When the sediment strength is very low, the water and sediment waves are in phase. On the other hand, high strength sediment behaves more as an elastic solid and there is a phase difference of 180°. In the actual situation with very large water waves, the sediment close to the mudline will be approximately in phase with the water

waves, and with increasing depth the phase relationship will vary continuously from fluidlike to solidlike. The bigger the wave, the more the sediment behaves as a fluid.

The complex interaction between the waves and the bottom movement can be quantified for a specific site if the sediment properties are known. As waves pass over a soft bottom, the sediment acts as a filter and removes some of the wave energy, thereby decreasing the wave height. A selective type of filtering probably occurs, depending on the amplitude and frequency of the wave components, but this has been studied for only a few selective locations using Fast Fourier Transform methods. The second advantage of the program is that it will calculate the distance required for the wave to degenerate 10% of its height, assuming that the wave period and the sediment properties remain constant over the calculated degeneration distance. If the sediment properties are known at the new location, then the degenerated wave can be further stepped in to the shoreline. There is some physical evidence for wave attenuation in the Mississippi Delta in the form of observations and one experiment. There is one location in the delta known locally as the "mudhole" where storm waves are visibly smaller than in the surrounding area. Fishing vessels caught in a storm will seek shelter in this zone of lower wave height until the storm abates. Undoubtedly, the sediments at this location are much softer than in the surrounding area. Shell Oil Company has conducted a measuring program, termed SWAMP (Sea Wave Attenuation Measuring Program), in which the sediment movements and wave heights have been measured over several miles distance from the Cognac platform in1000-ft water depth to a location in East Bay in 70-ft water depth. The few results that have been made available from this proprietary experiment indicate that wave attenuation over this distance during storm periods is much greater than would be calculated by rigid bottom wave theory.

3.2.3 Parametric Study of Sediment Movements. Since bottom movements are so dependent on the sediment properties and interaction between the waves and sediments, it is difficult to provide general solutions. As an illustration of movements and the possible effects on a bottom-resting structure, a brief parametric study was performed using actual data from a specific site in the South Pass Lease Block area of the Mississippi Delta. The sediment shear strengths at this location exhibit a typical profile found in the delta with a shallow crustal zone and a deeper cutback zone of weaker shear strength. The sediment properties are given in Table 3.2-1 as profile A. Two additional profiles were used: profile B is the same as profile A, but with all shear strengths reduced to 1/3 of the original, and profile C has all strengths increased by 3-1/3. The actual water depth at this location is 350 ft. This depth and two additional water depths of 240 and 460 ft were studied. The design storm wave for this location is a 50-ft wave with 10-sec period. Two additional periods of 6 to 18 sec were used. The wave lengths were computed using an option in the program which allows the calculation of the wave length compatible with the bottom conditions, the wave height and wave period. Two different G/S_u values were used: 32 and 100. This ratio, which relates the sediment stiffness to the undrained shear strength, has been found by experimental means to vary roughly within a range of 32 to 130 for sediments from the Gulf of Mexico. The smaller value seems appropriate for use after softening of the sediment by cyclic motion has occurred.

Figure 3.2-11 shows the effect of soil stiffness on the maximum horizontal movement at the mudline for profile A subjected to a 510-ft-long, 50-ft-high, 10-sec wave in various water depths. The effect of decreasing stiffness is to produce larger surface movements in the shallower water. At the 460-ft water depth, this particular wave will cause little movement, even with the less rigid soil.

The effect of changing the overall strength of the sediment is shown in Figure 3.2-12. The weak and the strong sediment profiles exhibit very nearly the same horizontal movements at the surface, regardless of the water depth. (Although not shown herein, the calculated subsurface movements were much greater for the weak sediment than they were for either of the

Table 3.2-1. Sediment properties used for parametric study.

Layer	Layer Thickness (ft)	Profile B Shear Strengths (psf)	Profile A Shear Strengths (psf)	Profile C Shear Strengths (psf)	Liquidity Index (li)	Initial N-Value (n_o)
1	15.00	16.70	50.00	167.00	1.000	0.089
2	25.00	50.00	150.00	500.00	0.700	0.054
3	25.00	20.00	60.00	200.00	0.830	0.069
4	55.00	113.00	340.00	1130.00	0.560	0.038
5	30.00	167.00	500.00	1670.00	0.540	0.035
6	30.00	200.00	600.00	2000.00	0.590	0.041
7	Infinite	1000.00	3000.00	10000.00	0.325	0.010

other two strengths, even though the surface movements were about the same.) The largest surface movements were produced by the medium strength sediment except in the 460-ft water depth.

In Figure 3.2-13, the effect of changing the wave period and length is shown. The 6-sec, 185-ft wave has virtually no effect on the sediment at the water depths shown. It would cause significant movement in shallower water if such a wave could exist. The 18-sec, 1600-ft-long wave will cause considerable movement in a water depth of 460 ft, primarily because the sediment is in resonance at this particular wave speed. The program would not converge for the shallower water depths, indicating that such a wave could not physically exist in the depths.

It should be noted that in an elastic material subjected to wave forces, the maximum shear stress occurs at a depth of 0.16 times the wave length below the sediment surface. If a soft cutback zone existed at approximately the same depth, the sediment movement would be greatly amplified due to the high imposed shear stress at the weakest zone.

As a further indication of both the bottom response during storm waves and the effect of bottom movement on a structure resting on the mudline, Figure 3.2-14 shows the vertical movements that would exist for the medium strength (profile A) sediment with G/S_u ratios of 32 and 100. Note that the phase relationship between wave height and vertical sediment movement varies with the difference in stiffness.

The above calculations were made for a bottom slope of 0.5%, which is a fairly typical slope for soft sediment areas of the Mississippi Delta. Calculations made within the program show that in addition to the horizontal cyclic movements shown in the above figures, there would be a downslope movement with a velocity of 0.48 ft/sec for G/S_u of 100 and 4.0 ft/sec for G/S_u of 32.

3.3 Slides in Loose Sands and Silts

Although the primary purpose of this report is to deal with cohesive sediments, the importance of submarine slides in cohesionless sediments is such that this brief discussion of their nature is included. Extensive slides have been recorded in marine deposits of loose sands, even though conventional slope stability analyses indicate the slopes should be stable. These movements occur on very flat slopes, nearly always less than 15°, and often on slopes of 2° to 3°. Static slopes in sands should be able to stand on slopes equal to their angle of internal friction, commonly about 28° to 35°.

Andresen and Bjerrum (1965) reported on two such slope failures, one in Trondheim Harbor that occurred in 1888 and the other in Helsinki Harbor in 1936. They also reported that there had been six major slides in Trondheim Harbor in the past 90 years, but there were eyewitnesses to only one (in 1888) at the time the paper was written. One feature common to these slides is that they propagated very rapidly, often in a matter of only a few minutes. This speed could only be reached if the sand lost all of its strength and moved as a flow slide.

The mechanism that initiated or triggered the slides is unknown in most cases. Retrogressive flow slides generally start as a result of local oversteepening due to erosion, and this may be a cause. It appears that many such slides occur during periods of abnormally low tides. This would result in larger than normal seepage pressures in the sediments. It is suspected that this was the cause of large slides recorded along the coast of Zeeland (Koppejan et al., 1948). Whatever triggers the slides, they progress retrogressively, slice by slice. As movement occurs, the sand liquefies as a result of the large strains in the loose material which, in turn, causes the development of large pore pressures. Regardless of where the slide starts, the liquefied sand will flow for significant distances. Movement usually stops when the sand reaches relatively flat areas or when the sand stratum extends to a point where it is covered by an overburden of sufficient thickness.

The studies by Andresen and Bjerrum (1965) showed that flow slides can occur in fine sands and in silts when their porosity is greater than about 44%. The grain size distribution of the materials also appears to have some importance, but this effect is not as clear as the porosity. In such loose materials, the act of sampling them will usually disturb the structure of the materials enough that the porosity measurements on samples will not be accurate. Accordingly, other methods of determining the porosity will be needed. Andresen and Bjerrum suggest small-scale blasting tests to determine susceptibility to flow slides, but they also suggest the use of the static cone penetrometer as a means of determining the relative density of sands in place without the difficulties of sampling.

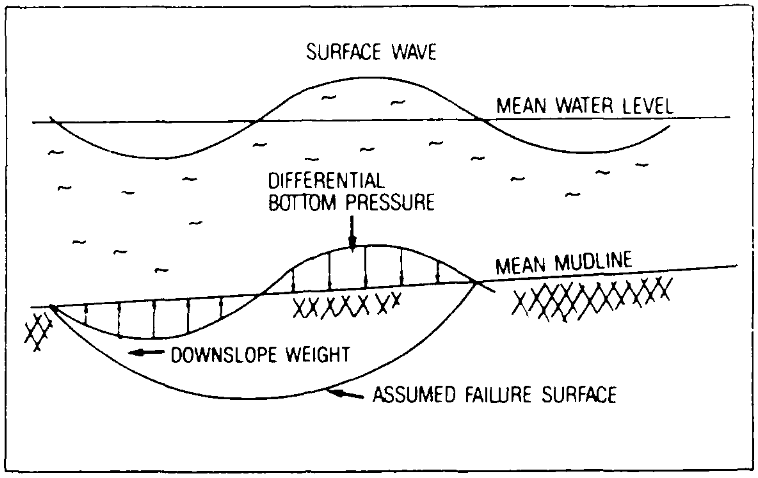


Figure 3.2-1. Effect of bottom pressures from storm-waves on bottom sediment movement (after Henkel, 1970).

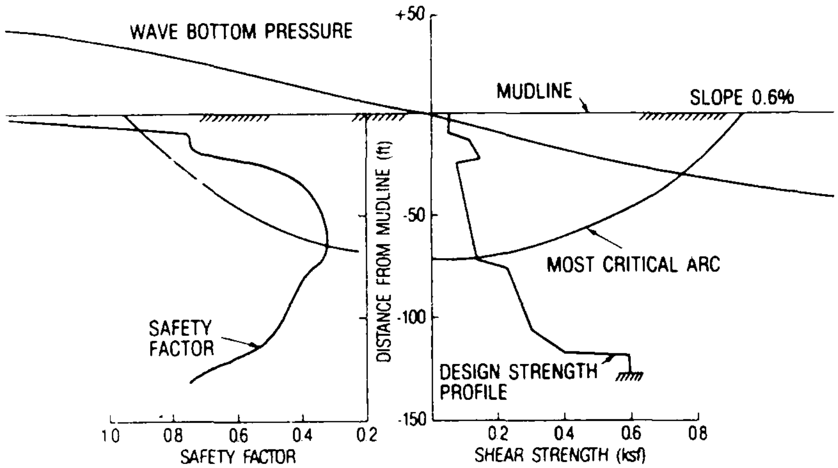


Figure 3.2-2. Illustration of conventional stability analyses to wave-seabottom interaction (after Kraft and Watkins, 1976).

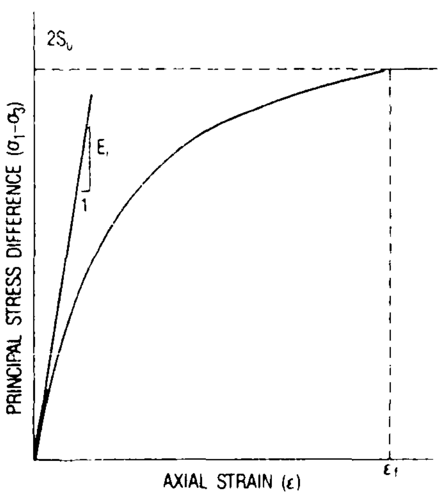


Figure 3.2-3. Hyperbolic stress-strain curve often assumed for sediments.

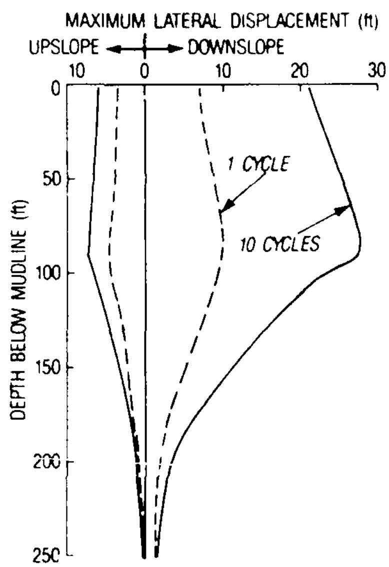


Figure 3.2-4. Influence of wave cycles on upslope and downslope movement (after Wright, 1976).

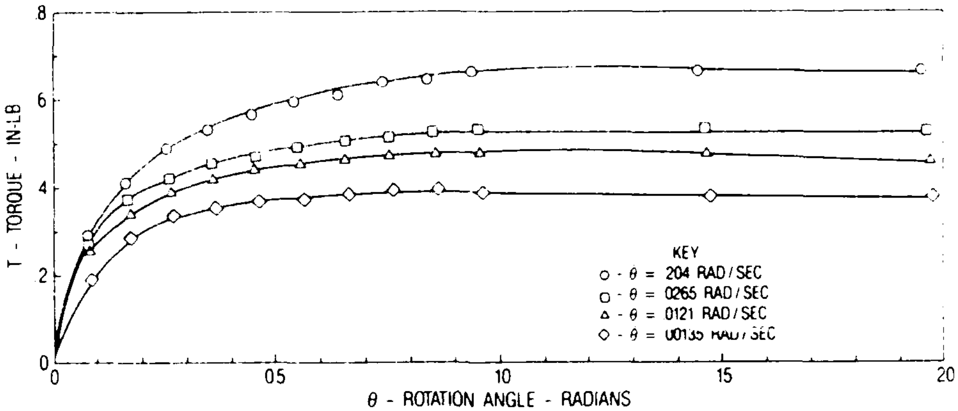


Figure 3.2-5. Typical torque-rotation angle test results from vane shear test (after Stevenson, 1973).

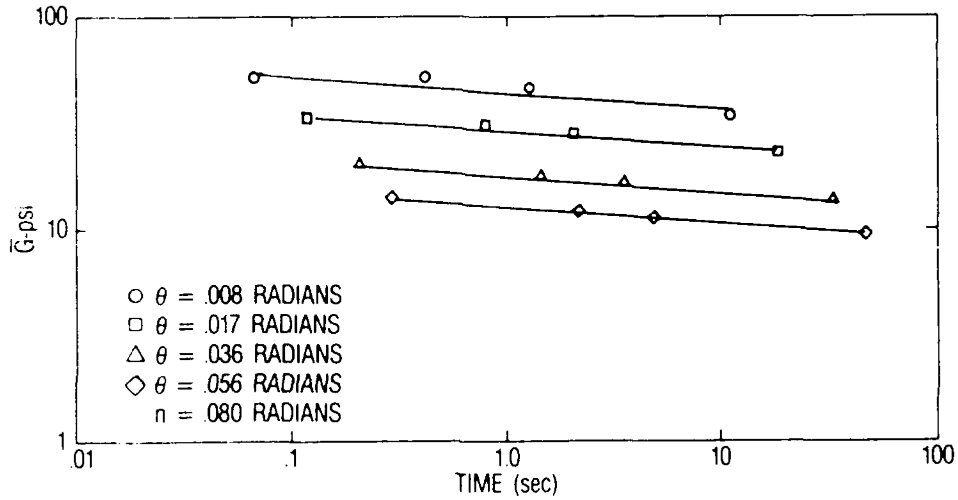


Figure 3.2-6. Plot of vane shear test results to obtain viscoelastic constants (after Stevenson, 1973).

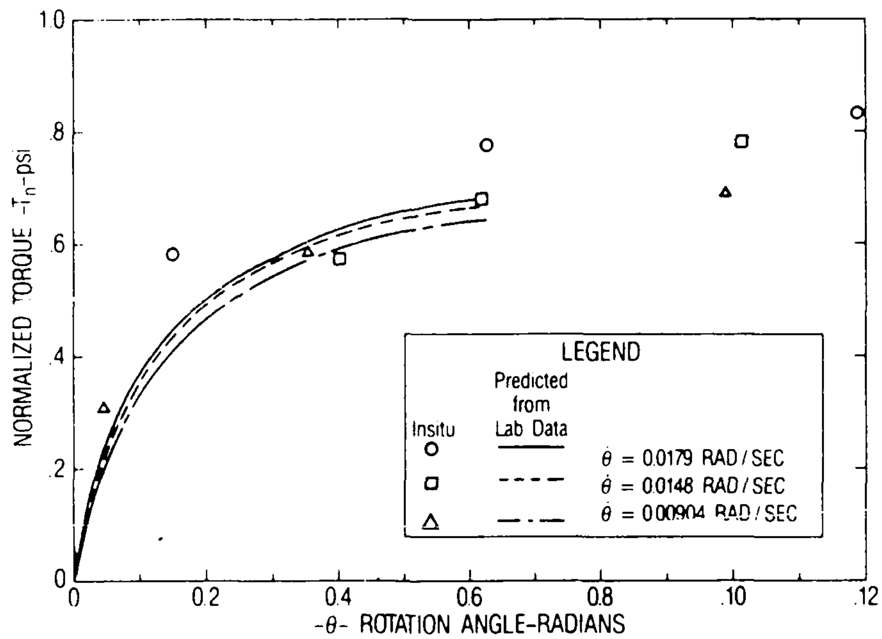


Figure 3.2-7. Comparison of lab predictions with in situ vane shear tests (after Stevenson, 1973).

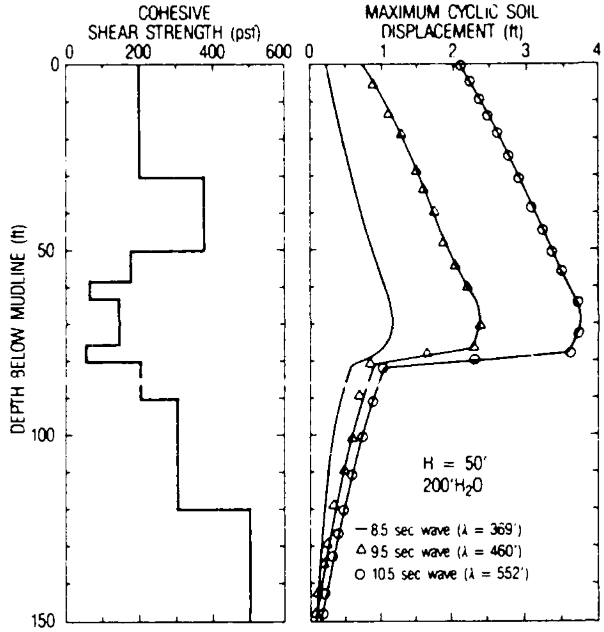


Figure 3.2-8. Illustration of cyclic displacement predicted by viscoelastic theory (after Schapery and Dunlap, 1978).

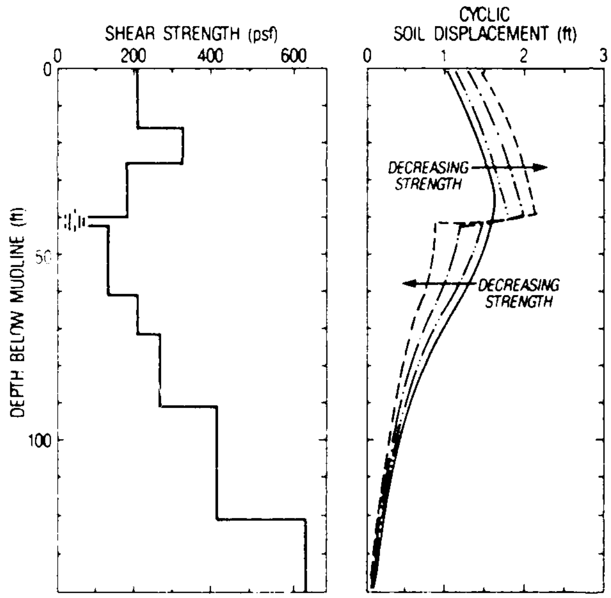


Figure 3.2-9. Effect of decreasing shear strength in "cutback" zone on cyclic displacement (after Schapery and Dunlap, 1978).

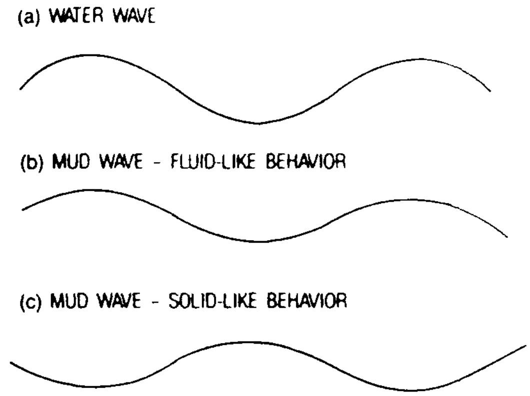


Figure 3.2-10. Response of bottom displacement to pressures from water waves (after Schapery and Dunlap, 1978).

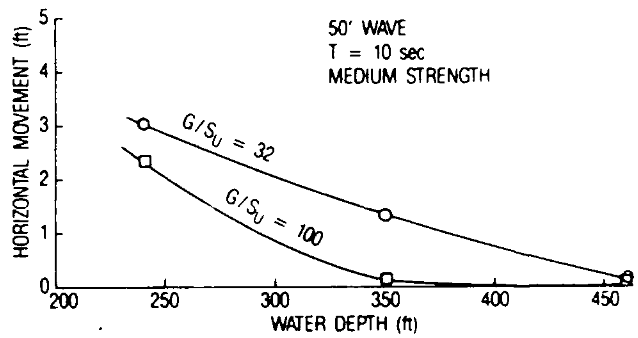


Figure 3.2-11. Influence of soil stiffness on maximum horizontal movement at mudline.

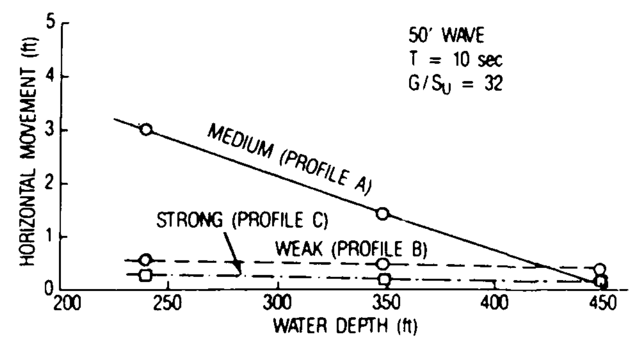


Figure 3.2-12. Influence of sediment strength changes on maximum horizontal movement at mudline.

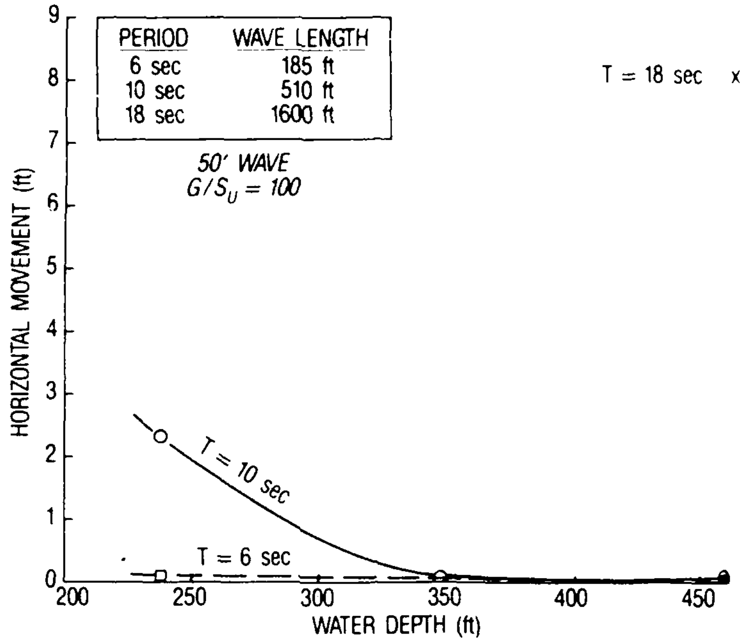


Figure 3.2-13. Influence of variations in wave length and period on maximum horizontal movement at mudline.

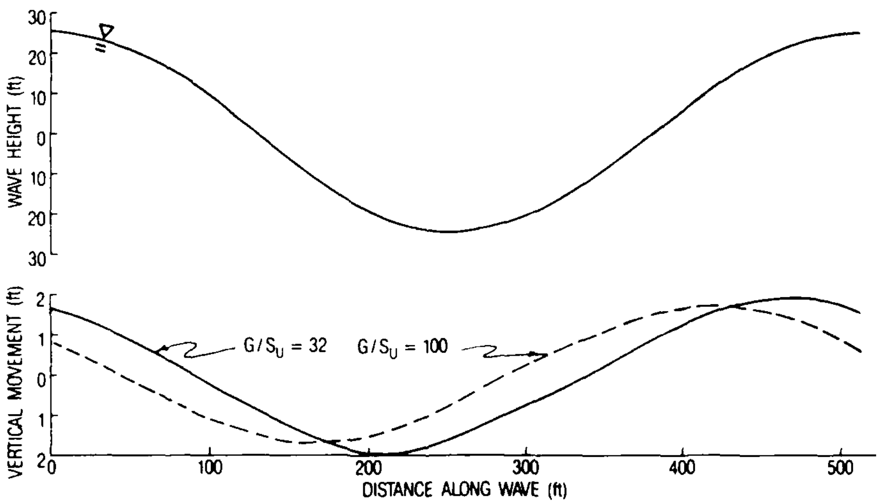


Figure 3.2-14. Seafloor vertical movements for 50-ft-high, 10-sec-period storm wave.

4.0 Scaling Effects

4.1 General

This section briefly reviews scaling effects, particularly as scaling relates to model studies of a bottom-resting structure. Scaling or modeling laws are used to predict prototype behavior from tests on models. Traditionally, this task has been difficult when dealing with materials that exhibit nonlinear behavior, such as soils. Under relatively light loads, which are often used in model studies, soils will behave in a linear, elastic fashion, whereas plastic behavior will be exhibited under high loads. In studies in which the sediment grain size is important, it can be nearly impossible to model the grain size and still maintain the behavior of the original material.

4.2 Scaling and Scour Effects

Scour represents a very complex three-dimensional flow and sediment transport problem involving many parameters. In the absence of field data upon which many design methods have been based, models are used. A severe limitation to the use of models lies in the uncertainty of how to include various scale values for time, force, and flow patterns. The effect of departures from exact scale values becomes proportionately greater as the scale of the tests is reduced.

Most tests made to evaluate scour around structures are concerned with fixed structures such as bridge piers, piles, submerged tanks, etc. A few tests have been published in which footings are allowed to move vertically (Teramoto et al., 1973), but the authors know of no tests involving lightly loaded structures that are capable of moving laterally while scour takes place, other than a few tests on model pipelines. However, even pipelines are constrained against large movements, and their shape does not fit with the problem at hand.

4.2.1 Factors to Consider in Scour. Hydraulic modeling involves two important factors, the Reynolds number, which governs movement of the sediment, and the Froude number, which governs the waves and currents. There is virtually no way to satisfy both the Reynolds and Froude numbers in the same model, i.e., by constructing a model in which the ratios of these numbers are the same as in the prototype. If the waves and currents are small enough in the model, then they will be unable to cause particle movement with the prototype material. Reducing the particle size usually results in a particle that is so small it has different behavior than the original material.

One way to improve the Reynolds number effect is to use lighter materials but with the same grain size. Materials that have been used in the same model are sand (2.65 specific gravity), crushed coal (1.4 to 1.5) and lucite (1.19). Obviously, it is not possible to model the actual grain shapes and characteristics with such diverse materials, and it has been found that the results from such a model are difficult to interpret.

An alternate approach is to use liquids with different viscosities in an attempt to maintain the Froude number. This requires unusual liquids such as glycerine, and often a liquid is not available that has the correct properties.

It has been found that different laboratories have usually found ways around the problem by developing their own empirical modeling approach based somewhat on theory and somewhat on prototype observations. An approach that has been used successfully for numerous scour studies at TAMU is to satisfy the Froude number, and get around the Reynolds number requirement for the soil by using different scales for the model structures. The obvious disadvantage of such an approach is the expense of constructing the models, since it is necessary to use two or perhaps three different size models to obtain results that are reliable.

4.2.2 Suggested Approach for Model Tests. An approach which could be used for the tests is as follows:

- Two, preferably three, scales of geometric models could be used to detect the trends in scale effects. At least three different ratios of wave height, H, to wave length, L, should be used.
- Two, preferably three, sediment specific gravities should be used with one scale model. Again, three different values of H/L should be used.

Scour depth and subsidence should be measured in the tests. The results could be plotted as demonstrated in Figure 4.2-1(a) and (b). The results could also be normalized in terms of the Reynolds number as shown in Figure 4.2-2(a) and (b), and also in terms of the Froude number (not shown). Similar plots could be made in terms of subsidence depth divided by water depth. These plots should allow extrapolation of the model tests to the prototype.

Such an experimental effort obviously involves numerous tests, but the lack of a complete modeling theory will require such an approach.

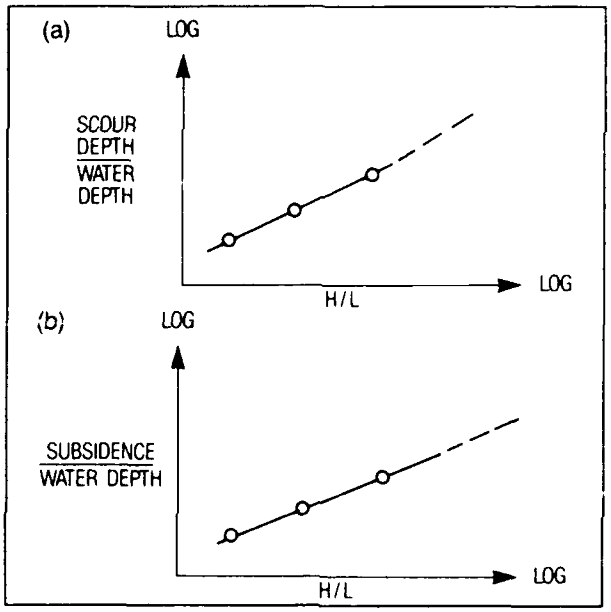


Figure 4.2-1. Probable results of model tests.

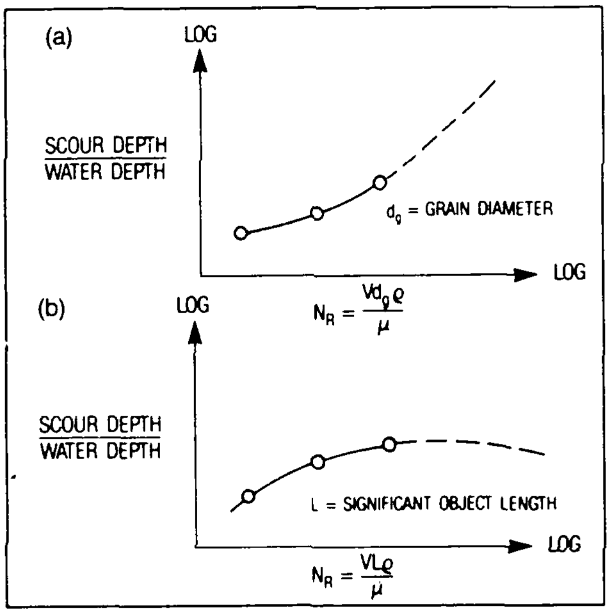


Figure 4.2-2. Plots of Reynolds number for model tests.

5.0 Recommendations

Mechanisms of soil-structure interaction that are not well understood should be investigated and evaluated in detail as a first step toward providing better predictions of behavior when only limited seafloor data are available. The following summary provides a brief description of work that should be initiated during the next few years. This additional work is highly recommended to support the planning of Navy operational scenarios.

• Dynamic bearing capacity and "shakedown" for structures rocking and sliding on the seafloor under the influence of waves and currents requires improved description and modeling.

— A solution of the common problem of soil-structure interaction in layered soils is required. At present, no adequate numerical solutions (models) of soil behavior are available for handling the dynamics for layered soils with respect to bearing capacity, penetration, and breakout. This is particularly true for seabottoms characterized by sands overlying soft clays.

— Analysis of field and laboratory data and model development to better predict structure motion and skidding on soft, weakly cohesive muds and on stiff clay bottoms should be conducted. Meager data are available on the undrained shear strength of the upper 6 inches of soft muds and on the strength characteristics of noncohesive, granular sediments with very light vertical (normal) loads. These soil characteristics strongly affect the performance of structures resting on the seafloor. Additional work is required to address the importance of rate effects on the resistance of a structure to movement on the seabed.

— The scaling effects, penetration rate functions, dynamic forces, coupling phenomenon, and strength degradation effects for punch-through and breakout in layered soils, including sands overlying soft clays, must be determined, along with the significance of these factors on predictive model development.

—The loading history of the structures on the seafloor, including current and wave loading, and uplift load during breakout, must be assembled and verified.

— Settlement of the structure on two or three different cohesive sediment profiles should be calculated for the refined loading history. These calculations should include the settlement due to remolding of the sediment under dynamic loading by the structure. Settlement estimates should be integrated with the breakout force estimates above to delineate those cohesive sediment profiles on which it is safe to place the structure.

— Soil-structure interaction of inclined and eccentric loads using scaled footings, evaluating the effects of repeated loads including the "shakedown" process, should be investigated. This should be a priority item.

• Assessment of the scaling factors involved in the process of scour for the subject footing(s) (specified geometry and bearing pressure) under the influence of waves and currents must be determined.

— The process of scour and fill about the structure on cohesionless seafloors, including the effects of sliding and plowing of sediments, must be evaluated to develop a prediction of depth of sand infill against the structure for various operating environments and scenarios. The potential for exceeding a sand infill height of 1.4 ft (factor of safety of 3 for pullout) must be known in order to determine if breakout from sand and gravel seafloors will be a problem.

— The structure foundation must be sealed in those areas in contact with the seafloor and in the sand transport zone to prevent the entry of sand into the foundation structure, as this sand fill seriously reduces the available uplift force for breakout. Assessment should be made of these factors.

• A technical assessment of the geology and soil types should be completed for all general operational areas. With this information some judgment and prediction of structure performance can be made. A knowledge of the soil types is essential in predicting the structure behavior and interaction with the seabed.

• The probable structure settlement/penetration on a range of seafloors (sediment types) should be quickly examined to identify those seafloor environments that pose an operational problem due to settlement and penetration. Also, those seafloor areas should be identified where it is desirable to deploy the structure, but not possible due to the cohesive sediment breakout problem. Given this identification, better assessment can be made of the desirability of improving predictive capability for breakout force and of the desirability of developing breakout aids on the structure, such as water jetting or electro-osmosis assistance.

• Investigation should be initiated to develop a working base of data on the properties of weakly cohesive submarine soils (e.g., the upper 6 inches of mud seabeds) and soil properties and strength characteristics of lightly loaded, noncohesive, granular soils.

• In situ instrumentation should be mounted on or deployed close to the bottom-sitting structure to aid in soil identification and dynamic soil behavior for situations where prior information is not available. State-of-the-art, in situ tools are presently available to make the types of measurements required.

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Dynamic Soil-Structure Interaction (S-SI) behavior on the seafloor describes the coupling of a structure and the seabed and their combined response to the influence of waves and currents and the material properties of the seafloor. S-SI problems are significant to the Navy when structures placed on the seafloor must be recovered promptly, maintain position (such as no lateral movement), maintain stability (such as not bury or tilt), and not be affected by abrasion and/or sedimentation. This report develops the critical marine geotechnical, geological, and environmental problems of S-SI for structure(s) placed on fine-grained sediments common to coastal areas. Only limited S-SI analysis of sand is presented in this report; however, significant additional work is seriously needed to support various naval operational scenarios requiring reliable predictive models. Detailed future research recommendations are provided herein and the ultimate success of the predictive models and operational strategies will depend critically upon the close integration of research by environmental scientists, geologists, and geotechnical engineers.

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